PROCEEDINGS
THE INSTITUTION OF CIVIL ENGINEERS

PART I
MARCH 1955

ORDINARY MEETING

16 November, 1954

DAVID MOWAT WATSON, B.Sc., President, in the Chair

The Council reported that they had recently transferred to the class of

Members

Clark, Anthony Henry, B.Eng. (*Sheffield*).
Fisher, John McNeal, B.Sc. (*Manchester*).
Mann, Adrian Bernard, O.B.E., B.Sc. (Eng.) (*London*).
Peregrine, Edgar Phillips, B.Sc. (*Wales*).
Wallis, Robert Graham, B.Sc.(Eng.) (*London*).

and had admitted as

Graduates

Advani, Chandru Tillumal, Stud.I.C.E.
Amner, David John, B.Sc.(Eng.) (*London*).
Ayton, Michael Charles, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Baker, Peter John, Stud.I.C.E.
Bazeley, Donald Bernard, Stud.I.C.E.
Benson, Frank Rupert, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Blundell, Christopher John, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Bottom, Keith Robert, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Bottomley, David Leslie, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Boustany, Faiz Wadie, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Bower, Joseph Cecil, B.Sc.Tech. (*Manchester*).
Bowler, Eric William, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Brace, Robin, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Bradley, Gerald Horace, Stud.I.C.E.
Broughton, Kenneth Henry.
Burton, George Bruce, B.E. (*New Zealand*).
Carlyle, William Johnstone. B.Sc, (*Belfast*), Stud.I.C.E.
Carter, Raymond Ivan.

Chin Kee Kheong, Stud.I.C.E.
Clatworthy, John Malcolm, B.Sc. (Eng.) (*London*).
Clements, Rex John, B.A. (*Cantab.*).
Cogill, William Henry, B.Sc. (*Cape Town*), Stud.I.C.E.
Collins, Philip Adolf, Stud.I.C.E.
Cottee, John Geoffrey, B.Eng. (*Liverpool*), Stud.I.C.E.
Cox, Oliver David Harry, B.A. (*Cantab.*).
Craig, Cameron, Stud.I.C.E.
Crowder, John Longden, B.Eng. (*Sheffield*).
Curtis, John, B.Sc. (Eng.) (*London*).
Curzon, Herbert Lewis, M.A. (*Cantab.*), Stud.I.C.E.
Davies, Edward Nathaniel, B.Sc. Tech. (*Manchester*), Stud.I.C.E.
Day, John Vernon, Stud.I.C.E.
Dearden, Eric Howard, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Demellweek, John, Stud.I.C.E.
Dewick, David Stanford, Stud.I.C.E.
Digges La Touche, Michael Cotton, B.A. (*Cantab.*).
Downey, Peter John, Stud.I.C.E.
Duncan, Raymond, B.Sc. (Eng.) (*London*).
Duncan, Ronald Gordon St Clair, B.Sc. (*Edinburgh*).
Easterbrook, Edward John, B.Sc. (Eng.) (*London*).
Fisher, Matthew Maxwell, B.Sc. (*Glasgow*), Stud.I.C.E.
Foster, Harold Alfred, B.Sc. (*Bristol*).
Fox, Norman Reginald, B.Sc. (*Bristol*).
Geach, Trevor Trenley Ward, Stud. I.C.E.
Gordon, Francis Alexander Murray, B.Sc. (*St Andrews*).
Gordon, Ronald Macgregor, B.Sc. (*Aberdeen*), Stud.I.C.E.
Gould, Peter, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Gower, Gordon Shields.
Graff, Samuel, B.Sc. (*Witwatersrand*).
Gregory, William John, B.Sc. (*Belfast*), Stud.I.C.E.
Gruber, Lajos Jeno, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Hadlow, John Minter, B.Sc. (*Nottingham*), Stud.I.C.E.
Harris, Alan Martin, B.Sc.(Eng.) (*London*).
Harvey, Keith Barnes, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Haseltine, Barry Albert, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Hind, Ronald William, B.Sc. (*Durham*).
Holloway, John McKinstry, Stud. I.C.E.
Horne, Donald Keith, Stud.I.C.E.
Horne, John Aitken, B.Sc. (*Edinburgh*), Stud.I.C.E.
Hudson, Norman Lionel, Stud.I.C.E.
Hunt, Alan Swinson, B.Sc. (*Nottingham*).
Ivey, David John.
Joglekar, Sharadkumar Mareshwar.
Jones, Anthony, B.Sc.(Eng.) (*London*).
Jones, John Stephen Major, B.A. (*Cantab.*).
Jones, Leonard Leslie, B.A. (*Cantab.*).
Jubb, Edward Michael, B.Sc. (*Birmingham*), Stud.I.C.E.
Lee Teck Chiow, Stud.I.C.E.
Luxon, Alec Martin, B.Sc.(Eng.) (*London*), Stud.I.C.E.
MacDowell, James Roughan Kenneth, B.A. (*Cantab.*), Stud.I.C.E.
Macky, Robert Graham, B.E. (*New Zealand*), Stud.I.C.E.
MacManus, Fintan, B.E. (*National*), Stud.I.C.E.
McQueen, Kenneth Reid, B.Sc. (*Glasgow*).
Mahfoz, Bin Khalid.
Malesa, John Francis, B.C.E. (*Melbourne*).
Marsh, Derek Leslie Donald, B.A. (*Cantab.*).
Marsh, John Vernon, B.Eng. (*Liverpool*), Stud.I.C.E.
Maskell, Alan David, B.Sc.(Eng.) (*Natal*), Stud.I.C.E.
Miles, George Edward, Stud.I.C.E.
Morrow, James Robert, B.E. (*New Zealand*), Stud.I.C.E.
O'Neill, Peter George, B.Sc.(Eng.) (*London*), Stud.I.C.E.
O'Rorke, Dermot Irwin, B.A. (*Cantab.*).
Parry, Richard Hawley Grey, B.C.E. (*Melbourne*).
Perry, Robert Joseph, Stud.I.C.E.
Phillips, John Haydn, B.Eng. (*Liverpool*), Stud.I.C.E.
Purvis, Grant Scurr, B.Sc. (*Durham*).
Randeniya, Munitunga, Stud.I.C.E.
Rees, William Eifion, B.Sc. (*Wales*).
Reeves, Peter, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Robertshaw, Ronald John.
Santa-Maria, Juan.
Schofield, John Derek, B.Sc.(Eng.) (*London*).
Simm, Keith Foster, B.Sc. (*Birmingham*).
Simpson, Howard Douglas, Stud.I.C.E.
Smith, Benjamin James, B.Sc.(Eng.) (*London*), Stud.I.C.E.
Smith, David William, Stud.I.C.E.
Smith, Hubert Brian, B.Sc.Tech. (*Manchester*), Stud.I.C.E.

Standish, Donald Wilbur, B.Sc. (*Durham*), Stud.I.C.E.
Stephenson, Alan Ronald, B.Sc. (*Nottingham*).
Stevens, Anthony, Stud.I.C.E.
Stott, Frederick Campbell, Stud. I.C E
Subramaniam, Sabaratnam.
Supersad, Jankie Nanan, B.Sc. (*Glasgow*), Stud.I.C.E.
Thomson, Harrold, Stud.I.C.E.
Vine, Joseph Peter, B.Sc.Tech. (*Manchester*), Stud.I.C.E.
Vivekananthan, Thambimuttu, B.Sc. (Eng.) (*London*), Stud.I.C.E.
Walker, Brian George, B.Sc.(Eng.) (*London*).
Weaks, George Geoffrey Cecil, B.A. (*Cantab.*).
Wilson, Antony William, B.Sc. (*Durham*), Stud.I.C.E.
Windebank, Keith Herbert, Stud I.C.E.

and had admitted as

Students

Akingbule, Olabanji Akinyomide.
Alford, Philip Leonard.
Amarasuriya, Amal Chandra.
Basham, Richard Harry Charles.
Beckett, Ronald Swanton.
Berwick, Peter Wood.
Biddle, Michael Richmond.
Billinghurst, Alan John.
Blight, John.
Bonner, Peter Ronald.
Buchanan, Donald Renfrew.
Burrough, Brian Laurence.
Chittenden, John Michael.
Cole, Kenneth Wilson.
Collier, David Michael.
Collingwood, Raymond William.
Copper, Edward Roger.
Cowley, Brian Ernest.
Datta, Santosh Kumar, B.E. (*Calcutta*).
Davies, Thomas Malcolm.
Evans, Garry Lewis.
Ford, Edwin.
Franklin, Ronald Eric.
Garsden, Henry Keith.
Gibson, Hugh Blyth.
Goodwin, Frederick George.
Greenwood, Roy.
Hart, Donald Alfred.
Hart, Richard William.
Hartley, Ian Malcolm.
Harrison, Robert Anthony.
Harvey, Ian Sherriff.
Hearst, John Knox.
Hogarth, Gordon.
Jackson, Edward John.
Jackson, John.
Jones, Dilwyn John.
Jones, Donald Ivor.
Journeaux, Clarence Raymond.
Laing, James Bruce.
Lawton, Robert Kershaw.
Lee, John Arthur Noël.
Maddock, David Victor.
Morbey, Richard Douglas.
Morris, David Malcolm.
Needham, George David.
Norton, Robert Dixon.
Owen, Peter James.
Peel, Edmund Anthony.
Plimmer, John Michael.
Prentice, Robin Laurence.
Ross, James.
Sethi, Prem Kumar.
Sinclair, Bruce Alan.
Thirumal, Paramalingam.
Thomson, Douglas Robert.
Vile, Gerald William David.
Welsh, George Gallacher.
Wendes, John.
Wijesena, Gunasekara Widana Mestrige.
Wootton, Paul Martin.
Wren, James David.
Wright, John Derek.

The following Paper was presented for discussion and, on the motion of the Chairman, the thanks of the Institution were accorded to the Author.

Paper No. 5978

"Present Trends in the Design of Pressure Tunnels and Shafts for Underground Hydro-Electric Power Stations"

by

* Charles Jaeger, Dr ès Sc. Techn.

SYNOPSIS

A number of underground power stations were built before the 1939–45 war, but nearly all of these were in the continent of Europe. The tremendous development of this type of design in all parts of the world is typical of post-war conditions. It is due partly to improvements in tunnelling technique and partly to the necessity for greater concentration of hydro-electric power in a few large stations. The Paper lists the types of underground power stations according to their characteristics and discusses the problems raised by these new structures.

After a short discussion on the hydrodynamics of underground stations, the main problems dealt with concern the design and construction of pressure tunnels and shafts. Strain and stress distribution in the rock and in the concrete lining, and stresses in the steel lining are analysed. The more modern theories are summarized, the danger of outside water pressure is stressed and the effect of rock grouting is discussed. The exposé leads progressively to an analysis of the modern ideas on pressure-tunnel design.

Underground power houses are, from a purely technical point of view, merely a paragraph out of a wider chapter on tunnel construction. Underground power houses are analysed from the point of view of overall dimensions, volume of excavation, design technique, and machinery.

INTRODUCTION

ALTHOUGH for many years there have been instances of underground hydro-electric power stations (the Brechbergmuehle plant in Germany was built in 1907, Porjus in Sweden in 1914), these have been the exceptions. This type of hydraulic structure is, however, becoming a classical design and power stations of the largest dimensions are now being designed and built underground.

A number of underground power stations have been connected to conventional pressure pipelines actuated above ground ; and, in several cases, lined pressure shafts have been connected to surface power houses. In the great majority of cases, however, underground power houses are connected to lined pressure shafts or galleries so that the simultaneous treatment of both in the same Paper is entirely justified.

In recent years the trend towards underground power stations has been accentuated. This is partly due to the improvements and developments in the general technique of tunnelling. These improvements have resulted in a lowering of the cost per unit volume excavated and a speeding up of construction.

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At the same time, methods for the design and construction of steel-lined shafts have been improving in the past 20 years. Methods have been evolved which have led to considerable savings in steel, as compared with ordinary pipelines. For example, it may be mentioned that at Etzel power station in Switzerland 6,300 tons of steel was required for the pipeline, whereas for Innertkirchen,[53, 54] which has a higher pressure and a similar length of pressure shaft carrying the same quantity of water, only 1,300 tons of reinforcement was required for the shaft.

In 1948 it was predicted that 500,000 horse-power would be the maximum transmission through a single steel-lined pressure shaft. That figure has already been exceeded at Kemano [77, 78] (Canada) and projects are being considered which will raise the maximum to 1,000,000 horsepower.

In spite of this high concentration of power in one installation, it can be stated that, on the whole, carefully planned underground power stations are safer than conventional surface designs.

Economic and financial advantages are at the root of these modern developments; but there are additional reasons for the popularity which this new type of power station has gained with experienced designers.

From the very beginning it was apparent that, in addition to the features already mentioned, the underground arrangement could have great flexibility. It is easy to adapt to any location, since it is practically unaffected by topographic or climatic conditions such as avalanches or extremely cold winters. Furthermore, the governing conditions can be improved with an underground station of good hydrodynamic design. In cases such as Clachan and Rheinau, where it is essential not to spoil the amenities of the landscape, subterranean power houses and pipes are the obvious choice.

Although it is considered that underground stations are bomb-proof, only in a few cases has this been a decisive argument in their favour.

The foregoing remarks explain why the underground station has recently been so much in favour with experienced engineers in Sweden, Norway, Italy, and Switzerland; and one of the great projects of Electricité de France now under construction—the Isère-Arc power station,[64] near Albertville, Savoie—is an underground station. In the case of the Roselend project in the Savoy Alps, where there is an available head of 1,171 metres, one of the most promising designs is that for an underground station. Other examples are the enormous Kemano scheme [77, 78] now under construction in Canada, north of Vancouver, and the no less important Bersimis scheme in the Province of Quebec.

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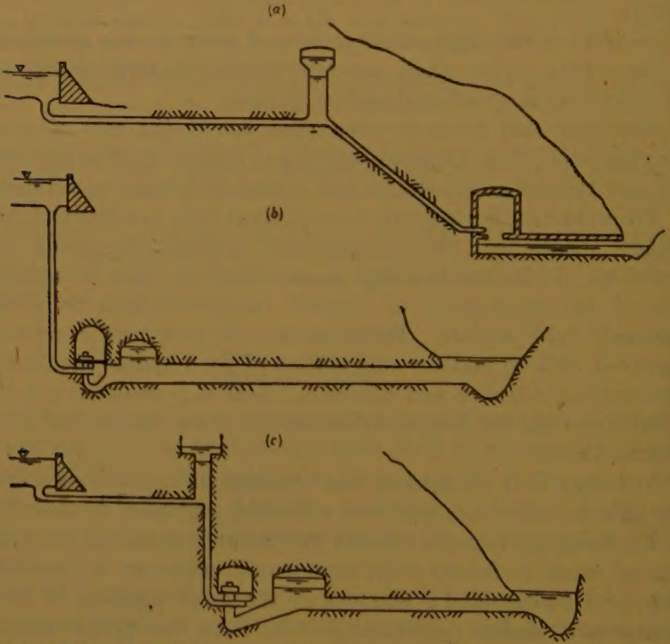
[53] The numbers refer to the classified bibliography on p. 170 *et seq.*

Existing technical literature adequately deals with aspects such as the general hydrodynamics of underground stations,[1, 2, 10, 11, 12] new tunnelling methods,[3, 4, 25, 26, 27] general design,[1, 2, 5, 6, 7] and ventilation of the power house,[6, 7] safety of the installation, and comparison of costs.[3, 4, 53, 55, 56] Furthermore, a considerable effort is now being made to reach a more satisfactory conclusion on some points concerning the design and the construction of the pressure tunnels and pressure shafts.

Types of Underground Power Stations

Underground power stations have sometimes been classified under "head developments" and "tail developments," according to whether the power house is at the head of the system, with a long tail-race tunnel, or at its downstream end, with a long upstream pressure conduit (*Fig. 1*).

Fig. 1



HEAD-RACE ARRANGEMENT (*a*), TAIL-RACE ARRANGEMENT (*b*), AND INTERMEDIATE SOLUTION (*c*)

The Swedish power stations (Krångede, Harsprånget, Hjälta,[59,60,61,62,63] etc.) are head developments. Innertkirchen in Switzerland, Isère-Arc[64] in France, Kemano[77, 78] and Bersimis in Canada, and most of the Italian underground power stations[5, 6, 7, 73, 74] belong to the tail development type. There are intermediate types where the conduits both upstream and downstream of the power house are of appreciable length.

Local conditions, particularly the general topography, constitute the main factor in the choice between the different types of power station. Unlined tail-race tunnels may be cheaper than lined pressure tunnels on the upstream side.

Another classification is based on the hydrodynamic characteristics of the scheme. In this classification there are the following five types (*Fig. 2*) :—

(1) The tail-race tunnel is a free level tunnel, without surge tank, downstream (Innertkirchen,[53, 54] Clachan, Krångede [57, 58]).

(2) The tail-race tunnel is a pressure tunnel with a conventional surge tank ; there is no surge tank on the upstream side (Harsprånget [59, 60, 61, 62, 63]).

(3) The whole system is under pressure, with surge tanks on both sides of the turbine. In this case, the behaviour of the system will depend on whether the turbine is of the reaction or impulse type. Several such projects are being seriously considered at present.

(4) Free flow exists in the tail-race tunnel during normal steady conditions but the tunnel will fill up during major disturbances in the flow conditions. In such a case the surge tank is called a "partial working" surge tank (Wettingen type [1, 2]).

(5) Finally the whole system, if very short, can be fully under pressure without any surge tank.

Examples can be quoted for each type of design. The advantages or disadvantages of any solution have to be discussed in relation to the scheme under consideration. The designer has to choose between : reaction or impulse wheel ; vertical or horizontal shaft ; main gates (or valves) to be in the main power-house chamber or in a separate chamber ; transformers underground or above ground.

The hydrodynamic aspects of underground power station design have already been dealt with extensively by the Author.[11, 12]

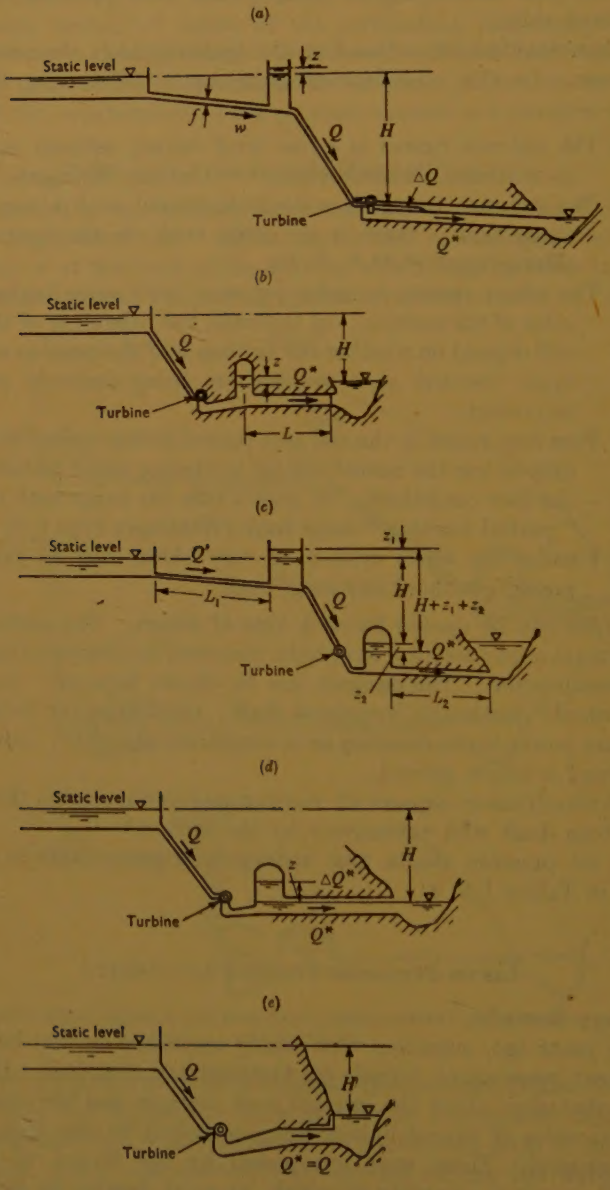
Data on pressure shafts and underground power stations are summarized in Tables 1 to 8.

LINED PRESSURE TUNNELS AND SHAFTS

Preliminary Remarks

Many years ago, engineers were vastly impressed by the bold design of the great trans-alpine tunnels (St Gothard and Simplon). Data were given at that time about the internal rock pressure and the creep of the rock ; examples of tunnel linings being squashed by the rock pressure were mentioned. These were interpreted by the theory of A. Heim (Zurich University) according to which, at great depths, its own weight causes the rock to behave as a plastic material or even as a liquid ; the

Fig. 2



FIVE PRINCIPAL HYDRO-DYNAMIC TYPES OF UNDERGROUND HYDRO-ELECTRIC POWER STATION pressure around any point is the same in all directions and equal to the hydrostatic pressure, the density being that of the rock.

When, some years later, engineers were requested to design the first large hydro-power pressure galleries in rock, or to submit proposals for the first steel-lined shafts, they were strongly influenced by the unfortunate history of the Ritom tunnel,[22] which had been damaged and dangerously fissured by internal water pressure.

Designers tried to develop theories explaining the strain and stress distribution in rock as a special case of the theory of thick cylinders. Not only did the two theories of rock deformation seem to have no connexion whatsoever, but they seemed to contradict each other. Another theory was being developed, assuming that an empty tunnel in the rock is subjected to a vertical field of parallel forces, so contradicting the theory of Heim and obscuring the issue. However, it can now be appreciated that the theory of Heim for empty tunnels and the calculations of the stresses and strains developed around a pressure tunnel are intimately connected. It is probable that future design of underground power stations will depend largely upon the designer's ability to reconcile the two theories and to take advantage of modern prestressing developments.

An interesting example is the design of steel-lined shafts. It is very likely that the first steel-lined pressure shafts were built with little knowledge of the actual stresses developing in the steel lining and in the rock. In the ensuing period, a rule-of-thumb method was used, according to which the stress σ in the lining was supposed to include the whole internal water pressure, neglecting any transmission of load to the rock, and should not exceed the yield stress of the steel plates. The steel lining was therefore calculated by means of the well-known formula for stresses in thin pipes using a high value for σ. This rule is still applied when the rock to be traversed is heterogeneous and somewhat unreliable. The linings of several modern shafts in the Italian Alps were calculated on similar assumptions (with $\sigma = 1{,}600$ to $1{,}800$ kilograms per square cm.). The effect of this rule is to reduce by more than a half the weight of steel to be used for the lining, as compared with the conventional pipeline of identical length, diameter, and pressure conditions.

Table 1 shows the stresses which would develop in the steel lining of actual shafts on the basis of this simplified assumption.

In the years 1925 to 1930, theories [20, 21, 23, 49, 50] were developed which took into account the relative elasticities of the steel lining, the concrete, and the rock, and considered the partial transmission of the water pressure from the steel lining to the concrete and the rock. The principle of this calculation is easily understood [49, 50] :—

> The water pressure p inside the conduit (*Fig. 3*) produces an expansion of the steel lining which is partially restrained by the concrete and the rock. If p_b and p_c denote the pressures transmitted

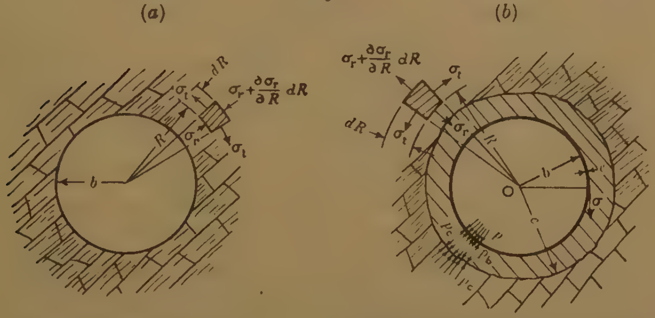
TABLE 1.—CHARACTERISTIC DATA ON SOME GREAT STEEL-LINED PRESSURE SHAFTS

Power station	Country	Gross static head :		Internal pipe diameter :		Steel thickness :		Stresses * in steel plate :	
		m.	ft	m.	ft	mm.	in.	kg./sq. cm.	lb./sq. in.
Handeck I	Switzerland	550	1,800	2·10	6·89	20	0·79	2,885	41,000
Innertkirchen	"	672·3	2,200	2·40	7·87	20	0·79	4,030	57,400
Innertkirchen test tunnel	"	1,500	4,920	2·20	7·22	10	0·39	16,500	234,000
Handeck II	"	463	1,520	2·15	7·05	12	0·47	4,150	59,000
Oberaar shaft	"	531	1,740	1·65	5·41	13	0·51	3,380	48,000
Oberaar free pipe	"	532·5	1,743	1·40	4·59	28	1·10	1,330	18,900
Wassen	"	279·2	915	2·40	7·87	11	0·43	3,050	43,100
Gondo	"	490	1,605	1·6	5·25	17	0·67	2,300	32,700
Verbano	"	297	974	2·8	9·19	26	1·01	1,600	22,800
Piage-de-Vizille	France	142·5	466	3·5	11·48	11	0·43	2,270	32,200
Isère-Arc	"	152	500	3·7	12·14	16	0·63	1,750	24,900
Kemano	Canada	790	2,597	3·35	10·99	49·3	1·95	2,670	37,900
Pfaffensprung (projected)	Switzerland	818·5	2,680	2·50	8·20	34	1·34	3,000	42,500
Abjora	Norway	442	1,450	2·30	7·55	22	0·87	2,310	32,900
Lyse	"	630	2,067	2·0	6·56	18	0·71	3,500	49,700
Tafjord	"	396	1,300	1·75	5·74	15	0·59	2,410	34,300
Rössaga	"	245	804	2·8	9·19	23	0·91	1,500	21,350

* At the bottom of the shaft, assuming that no load is transmitted to the rock.

in a radial direction from the steel to the concrete and from the concrete to the rock respectively, then tangential stress σ in the steel lining is $\sigma = (p - p_b)b/e$, where b denotes the radius of the steel lining and e its thickness. If E denotes the modulus of elasticity, then the elastic radial displacement of the lining is $\Delta b = \sigma b/E$, which must be equal to the radial displacement of the concrete. A second system of equations is written for the radial displacements of the concrete and of the rock, which must also be equal. The two systems of equations yield the values of p_b and p_c, and the stresses σ_r (radial) and σ_t (circumferential or tangential) at a distance R from the centre O.

Fig. 3


(a)
(b)
dR
σ_t
$\sigma_r + \frac{\partial \sigma_r}{\partial R} dR$
R
σ_r
σ_t
b
$\sigma_r + \frac{\partial \sigma_r}{\partial R} dR$
σ_t
dR
σ_t
σ_r
R
b
O
σ
c
p_c
p
p_b
p_c

STRESS DISTRIBUTION IN ROCK AROUND AN UNLINED PRESSURE TUNNEL

STRESS DISTRIBUTION BETWEEN STEEL LINING, CONCRETE, AND ROCK IN STEEL-LINED PRESSURE GALLERIES

The calculation shows that for small water pressures p (up to 5 kg. per square cm.) and small diameters D (2·5 to 3·5 metres) the tensile stress σ in the steel lining is low, even when the steel thickness e is small.

Many measurements of stresses in steel linings have confirmed this conclusion and so proved that the bulk of the load is taken by the rock, as foreseen by the theory. If the pressure or the diameter increase, the proportion of the load taken by the steel lining also increases.

Another conclusion is that circumferential tensile stresses σ_t develop in the rock. The calculation also shows the great importance of temperature stresses and of any gap existing between steel lining and concrete lining before the tunnel or shaft is put under pressure.

Because of these tensile stresses in the rock, it was thought that there would be a limit to the dimensions of steel-lined pressure shafts. Theories regarding what would happen if radial fissures were to develop in the rock were propounded but were not convincing.

In contradiction to the theory, tensile stresses developing in the rock seem to have no effect on the stability of the lining ; rock which should be fissured remains solid and doubts may therefore arise with regard to the validity of the theory.

Up to the present no failure of steel linings arising from the development of rock fissures has been reported. But failures of the lining have been caused by external water pressure, by grossly defective design, or by lack of homogeneity of the concrete filling between the steel lining and the rock.

A direct proof that the strained rock behaves quite differently from what could be expected from the theory is given by the behaviour of three Italian pressure shafts designed as early as 1925/1926 by Marinoni, the characteristic data of which are given in Table 2. These three shafts are lined with corrugated steel plates, the thickness of which is less than 4 mm.; these cover an ordinary concrete lining. The stations have been in operation for 25 years and the accidents which have occurred to the shafts have all been caused by outside pressure and were quickly repaired. This experience shows that in some cases the whole hydrostatic load can be transmitted to the rock, the steel lining serving only as a watertight sheath.

TABLE 2.—SHAFTS DESIGNED BY MARINONI

	Head:		Diameter of shaft:		Rock	Theoretical value of σ_t in rock:	
	m.	ft	m.	ft		kg./ sq. cm.	lb./ sq. in.
Pallanzeno	527	1,730	1·80 to 1·50	6 to 5	Gneiss	— 52·7	— 750
Rovesca	720	2,360	1·20	4	Schists and gneiss	— 72·0	— 1,025
Mese	758	2,490	1·80	6	Gneiss (not compact)	— 75·8	— 1,075

The theoretical values calculated for σ_t in Table 2 are higher than those which would normally be assumed for a homogenous rock and much higher than for a fissured rock.

Clearly, therefore, the ability of rock to withstand the tensile stresses σ_t is far greater than was expected. As a result new theories had to be evolved on the basis of extensive research and measurements, and these are of great importance for the technique and design of pressure shafts.

Most of this new research work assumes, either explicitly or implicitly, the validity of the old theory of Heim concerning the internal compression stresses already present in rock prior to any excavation. This rock pressure, which at some greater depth may be approximately the same in any direction at any particular point, is, in the mind of the designer, equivalent to a prestressing of the rock. This prestressing may more than compensate the tensile stresses induced by the water pressure in the tunnel.

A tunnel or shaft will be safe if the resultant stresses at any point of the steel lining or at any point of the rock, including the effect of prestressing, are well inside the known permissible values.

These introductory remarks on the theories of rock behaviour will make it easier to present the more detailed analysis of the complicated conditions prevailing in lined and unlined pressure tunnels and shafts.

The next logical steps in the discussion of this subject are consideration of the constructional materials available and an analysis of the behaviour of concrete, steel, and rock. Rock is by far the most important material and very little is yet known about its behaviour in situ.

Concrete for Tunnel Linings

Only vibrated concrete is now used for filling between the steel lining and the rock. This filling must be homogeneous, without voids, and as impervious as it can be made.

The technique of vibrating the concrete is the same as that which has been developed for dams. In order to be sure that the filling is as perfect as possible, there must be a limit to the length of the pipe sections used for lining, and special care has to be taken with the anchorage. Table 3

TABLE 3.—THICKNESS OF CONCRETE PACKING, LENGTH OF PIPES, AND DESCRIPTION OF ANCHORAGES FROM STEEL TO CONCRETE

Pressure shaft	Thickness of concrete:		Length of pipes:		Anchorages
	m.	in.	m.	ft	
Innertkirchen	?	?	10–12*	33–39	None
Handeck II †	0·06	2·33	?	?	None
Oberaar †	0·10–0·13	4–5	10	33	60/20 to 80/25-mm. angles. Six on a length of 10 m.
Verbano	0·15	6	10	33	
Gondo	0·2–0·35	8–14	8–12‡	26–39	None
Isère-Arc	0·40	16	3	10	100/100/10 to 120/120/12-mm. angles. Two hoops per 3 m.
Kemano §	0·6	24	2 × 4·3 or 2 × 8·6 welded	2 × 14 or 2 × 28 together	Prepakt concrete, 1–3 in. placed by 10″ pipe.
Proposed by Mr Talobre for new designs	0·25	10	—	—	Hedgehog hair

* Owing to insufficient rock overburden, the distribution pipes of Innertkirchen are enclosed in a second pipe, which has a wall thickness of 20 mm., the clearance between the pipes being 20–40 cm.

† Concreting pipes were used. (The concrete for Gondo was a rich mix, with 275 kilograms of Portland cement and 50 kilograms of pozzolanic cement per cubic metre of concrete. An air entraining agent was also added.)

‡ On the horizontal section the pipe length was restricted to 8 metres because of limitations on the weight of sections of pipe.

§ " Prepakt " concrete, aggregates 1–3 inches, placed by 10-inch pipe. Two lengths of pipe were welded together and the assembled sections then brought to the inclined shaft.

gives some examples of the thickness of the concrete filling, the pipe lengths used, and types of anchorage.

Extensive use has been made of the concrete-pump on several shafts. The new concreting technique used at Kemano is well worth mentioning. After considerable study, the designers determined that better control would be afforded by using the Prepakt method. Point-to-point contact of the rock and aggregates minimizes the contraction of the concrete.

The aggregates, ranging from 1 to 3 inch size, are chuted into the space behind the steel liner (a space with an average width of 2 feet) through a 10-inch-diameter pipe, which is kept full at all times to minimize breakage of the stones.

The grout mixture includes cement, fine sand, an air-entrainment mixture, and a wetting agent. It is pumped into the aggregates through grout nipples placed in the pipe during fabrication. Four holes are arranged around a horizontal plane and the grout is pumped into these holes until it emerges from holes at a level 7 feet higher. Then the grouting equipment, which is mounted on three interconnected platforms, is moved up.

At the Maggia pressure shaft the wet concrete, containing an excess of water and sand, slides down small inclined channels a distance of about 500 metres. The excess of sand avoids segregation of the concrete, which, when set, is watertight but has a low crushing strength.

It will be seen from Table 3 that in some cases the steel linings are reinforced by steel hoops and anchored in the concrete. The designers (OFINCO, Geneva) of the Gondo power station definitely rejected any anchorage because of the danger of producing voids in the concrete.

During the concreting process at Gondo, the steel lining was reinforced by internal strutting. Concreting of the section of the shaft not lined with steel was carried out by means of a movable braced shutter, 4 to 5 metres long, moving at 30 to 50 centimetres per hour. Concreting was continuous by day and night, including week-ends.

Other designers insist that this anchorage is necessary to prevent buckling, when the pipe is emptied, owing to the external water pressure arising from seepage through the rock. Some designers have produced interesting proposals to avoid the formation of voids at the anchorage hoops and to facilitate the filling of them when grouting between the steel lining and the concrete.

It has recently been proposed to use expanding cement for the concrete packing of the steel pipes.

Steel for Tunnel Linings

The steel most commonly used for the lining is ordinary steel, as can be seen from Table 4.

At Oberaar the lower part of the conduit passes underneath the Grimsel reservoir basin. Since the overburden is comparatively shallow, the rock

TABLE 4.—GRADES OF STEEL USED FOR LINED SHAFTS : COMPARISON WITH BRITISH PRACTICE FOR PIPELINE DESIGN

	Ultimate strength :	
	kg./sq. cm.	tons/sq. in.
Steel commonly recommended in Great Britain for pipelines :		
B.S. 15, structural steel	4,300–5,100	28–33
B.S. 14, marine boiler plate	4,300–5,000	28–32
B.S. 534, water and gas lap-welded pipes	3,700–4,300	24–28
Malgovert : prestressed pipe	5,600	36·5
Glockner-Kaprunwerke	4,400–5,800	28·5–37·5*
Innertkirchen	4,300–4,800	28–31
Oberaar :		
Ordinary lining of S.M.I. quality	?	?
Cr.–Cu.-steel for free pipe section	5,000–6,000	32–39
Gondo :		
Ordinary steel A.35	3,500–4,200	23–27
Special steel A.42	4,200–5,000	27–32
Kemano :		
$e < 1$ inch, A.S.T.M. A.285 fire-box quality grade B		
$e < 1$ inch, A.S.T.M. A.201 fire-box quality grade A		

* Steel containing 0·02 to 0·08 per cent aluminium $\sigma_{elas} = 0{\cdot}65$ to $0{\cdot}68 \times \sigma_{ult}$.

cannot withstand the high bursting pressures. For this section the lined shaft is replaced by a steel pipe lying free in a tunnel so that no stress can be transmitted to the rock. For this stretch of free pipe the steel used is a Cu.–Cr. steel with an ultimate strength of 50 to 60 kg. per square mm. (32 to 39 tons † per square inch). The diameter is 1·40 metre and the steel thickness 28 mm. ; therefore, the stress in the steel pipe for hydrostatic load (disregarding the pressure gradient) is 1,330 kg. per square cm. (see Table 1). All pipe sections were tested at works. This pipe was both welded and stress-relieved at site. A special coupling has been designed to connect this special steel pipe to the ordinary steel plates used for the lining of the shaft. This is the first application of special alloy steel for pressure shafts and it may lead to further developments.

The welding of transverse joints inside a shaft is a difficult operation. Radiographic checking of the welds is now becoming a routine technique at several sites. Isotopes of irridium (192) and cobalt (60) were used at Oberhasli ; cobalt was used at Kemano.

Tables 1 and 5 summarize the data available for some of the large steel-lined shafts now in existence and these can be compared with the figures given in Table 4. Table 3 gives the lengths of the pipe sections used at different places.

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† Avoirdupois.

Rock

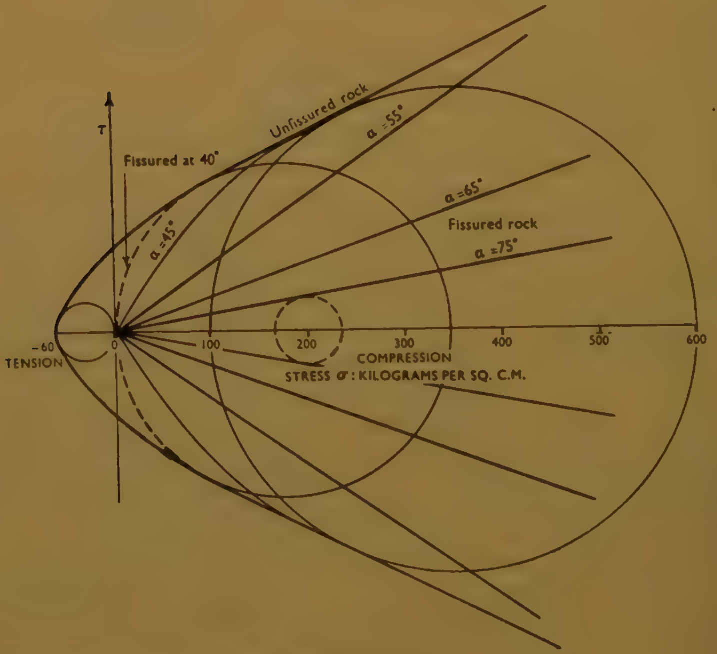
Natural rock is the most important material in steel-lined pressure shafts since it carries most of the pressure loading. Being well aware of visible or assumed faults in rock, civil engineers show some hesitation in permitting it to be subjected to a high proportion of the water-pressure stresses.

The rock properties which are important in the design of steel-lined shafts are its crushing strength and tensile strength, its modulus of elasticity, and its imperviousness. But, above all, the natural prestressing of the rock provides the key to its behaviour.

Crushing Strength of Rock

The crushing strength of sound rock as ascertained from unfissured samples is often very high. Values of up to 2,000 kg. per square cm. have been obtained from Urseren granite (Swiss Alps). The crushing strength is greatly increased if a transverse strain is applied. *Fig. 4* (after Talobre[25, 26, 27]) shows the Mohr circles corresponding to a rock sample with a crushing strength σ_{max} of 300 kg. per square cm.; when a transverse stress component $\sigma = 100$ kg. per square cm. is applied, the crushing strength rises to 600 kg. per square cm., although the same rock is said

Fig. 4



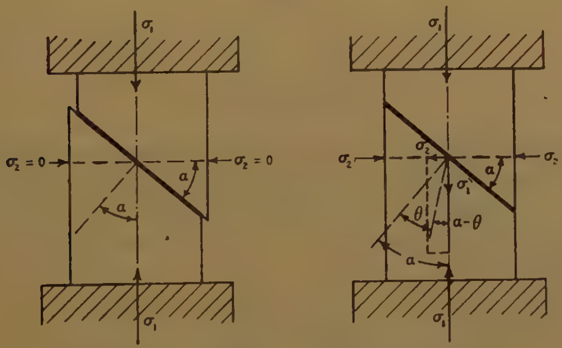
MOHR CIRCLES FOR ROCK

to have a tensile strength of 60 kg. per square cm. It is possible to trace, for varying conditions of transverse strain, the envelope to all the Mohr circles ; it is a curve showing the final strength of the assumed homogeneous and unfissured rock.

If the rock is fissured, then its crushing strength is not decreased so long as the angle $\frac{1}{2}\pi - \alpha$ between the direction of the compression stress σ_1 and the direction of the fissures is such that $\alpha \leqslant \phi$, where ϕ is the angle of friction between rock and rock. Obviously, $\alpha = \phi$ is a limit value at which the conditions of equilibrium of the sample will change. If $\alpha > \phi$ and if no strain component acts in a direction perpendicular to σ_1, the sample shows no resistance to the compression force.

When a stress σ_2 perpendicular to σ_1 is introduced, then, according to *Fig. 5*, the angle θ of the stress ρ resulting from σ_1 and σ_2 must be $\theta \leqslant \phi$. Then $\sigma_2 = \sigma_1 \tan(\alpha - \theta)$ and therefore $\sigma_2 \geqslant \sigma_1 \tan(\alpha - \phi)$.

Fig. 5



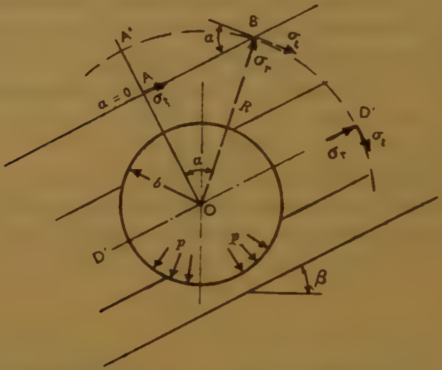
LATERAL STRESS IN FISSURED ROCK

In *Fig. 4* it is assumed that $\phi = 40$ degrees and the curves for the limiting crushing strength for $\alpha = 45$, 55, 65, and 75 degrees respectively are traced using Coulomb's law for combined shear and friction.

When an unlined pressure gallery is submitted to an internal pressure p, the resulting circumferential stress σ_t developed anywhere in the rock at a distance R is always equal to $\sigma_t = -\sigma_r$, where σ_r is a radial compression stress whilst σ_t is a tensile stress. $\sigma_r = -\sigma_t = p$ for $R = a =$ radius of the gallery ; σ_r and σ_t are principal stresses. In *Fig. 6* the line OAA′ is assumed normal to the direction of the strata, which has an inclination $\alpha = 0$. If the point B′ moves from A towards infinity, the angle α increases and—when neglecting the overburden—the resultant stress has a tendency to open the fissures as soon as $\alpha = 45$ degrees, the tendency being obviously to a maximum along D′D′.

If the rock is of poor quality or if the overburden of the rock above the unlined tunnel is not sufficient, cracks may occur along D′D′ and water may seep through the fissures. Cases of severe damage are known, especially where landslides have been caused by the seeping water.[22, 25, 40, 62]

Fig. 6



Pressure gallery in inclined fissured
rock strata

The " overburden-action " will keep the fissures closed and check the detrimental effect of the tensile stresses irrespective of the angle β of the strata to the horizontal. The explanation of this action requires additional investigation and measurements, the results of which have only recently become available.

Elasticity of Rock

In 1923 the pressure tunnel of Ritom in Switzerland was severely damaged by cracking, attributed to yielding of the rock.[22] Measurements were made in the tunnel of Amsteg, in order to determine the modulus of elasticity of the rock in situ. These measurements were probably the first to be published on this subject. Later on, similar tests were repeated in many tunnels [31] (Lucendro, Malgovert, Rossens, Italian tunnels, etc.), sometimes in conjunction with the construction of large dams, but mainly in order to get first-hand information on the behaviour of pressure tunnels. The results obtained showed that the modulus of elasticity E depends not only on the rock type and quality but also on the direction of the force T, E being smaller in the direction perpendicular to the strata than in the direction parallel to the strata. The measured values in both directions are :—

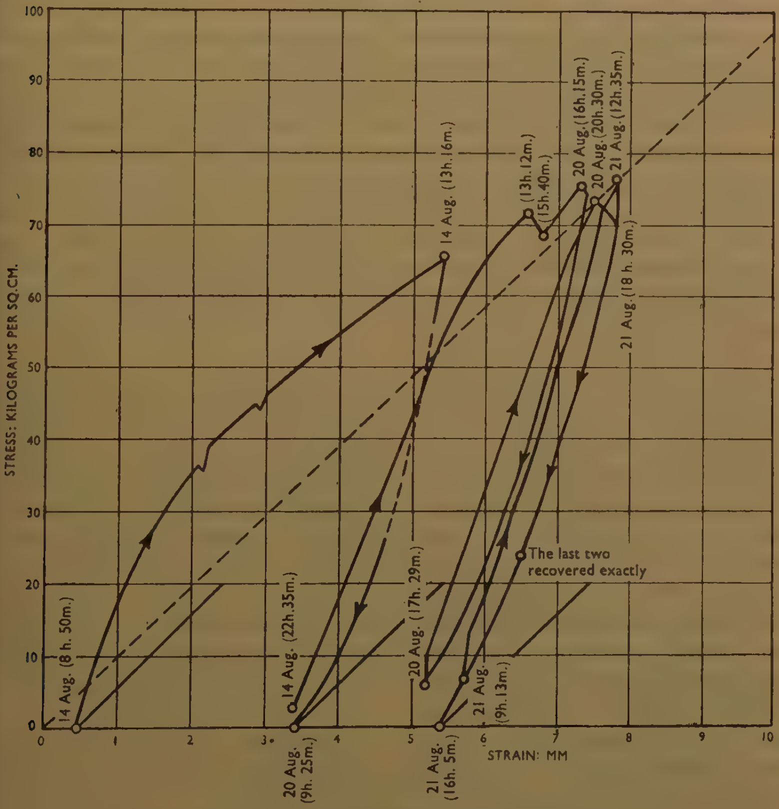
	Normal to strata : kg. per sq. cm.	Parallel to strata : kg. per sq. cm.
Rossens sandstone . .	22,000	50,000
Urseren granite . . .	250,000	400,000
Urseren schists . . .	80,000	280,000

There is an interesting comparison between these figures and the value of *E* measured at Chimbote in Peru, which is 350,000 kg. per square cm. More recently P. Bernard measured 66,000 to 190,000 kg. per square cm. for schists, 250,000 to 455,000 for quartzites, and only 2,000 to 26,000 for sandstones.

Apart from the very considerable differences between the values for a hard homogeneous rock like granite (Urseren) and a good compact sandstone (Rossens), the importance of the direction of measurement relative to the strata is obvious. It can be assumed that the lower value of *E* (measured normal to the strata) is attributable to the plastic deformation of the fissures, which first have to be closed before elastic deformation becomes possible.

If a cycle of loading, ranging from zero to maximum, is repeated several times, then the plastic deformation can be separated from the elastic rock deformation, as shown by *Fig. 7*. This diagram was derived by Talobre

Fig. 7



PLASTIC AND ELASTIC ROCK DEFORMATION IN MALGOVERT TUNNEL (AFTER TALOBRE) from tests inside the Malgovert tunnel,[25] where the pressure was raised to 60 to 75 kg. per square cm.

After five applications of the loading cycle in a period of 7 days, the rock was behaving elastically. The modulus of elasticity was then nearly equal to the so-called "dynamic modulus" as obtained by laboratory tests on a dense rock sample using acoustic methods. Similar diagrams were published by Oberti[43, 44, 45] as early as 1944, in a research Paper of great importance, in which much pioneering research work was first made available to engineers.

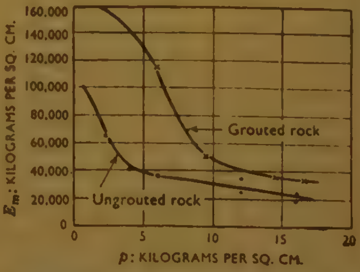
The experiments by Gicot at Rossens yielded similar results.[31, 36] Frey-Bär carried out measurements[31, 38] on several stretches of the Lucendro tunnel, in order to test different types of gneiss. The hydrostatic pressure used for the tests was about 10 kg. per square cm. Both rough and lined sections were tested. The method of testing involved measuring the length variations of two mutually perpendicular diameters, and was similar to the method used years earlier in the Amsteg tunnel. The measuring apparatus was of the Gerber type built by Huggenberger.

It was found that, in so-called Para-gneiss, the overall modulus E', including plastic deformations, was $E' = 120{,}000$ kg. per square cm.; whereas the elastic modulus was $E = 390{,}000$ kg. per square cm.

A concrete-lined section of the tunnel with steel reinforcement bars proved to have a value of $E' = 150{,}000$ to 230,000 kg. per square cm., 20 to 28 per cent of the strain being plastic deformation of hysteresis type.

The technique used by Oberti[31, 37] in badly fissured limestone was very similar to that used by Frey-Bär. He observed a high proportion of plastic deformation at the beginning of the tests and found that E was decreasing with increasing pressures. Grouting of the fissured rock definitely improved the modulus E. He found that, for a pressure p (*Fig. 8*) between 8 and 15 kg. per square cm., E varied from 35,000 to 52,000 kg. per square cm. before and after grouting.

Fig. 8



Effect of grouting on the modulus of elasticity of fissured rock (after Oberti)

Quite independently, Bernard[17] made further similar tests, measuring the strains in tunnel linings during and after injection and determining the stresses before and after grouting. His tests showed that the lining contracts during the grouting process (to a great extent permanently); the stresses under loading are reduced and are equalized in the various planes; the deformations measured in the earlier tests were almost eliminated by the grouting.

Tests in schists were carried out with grouting pressures of 30 kg. per square cm. (absorbing about 175 kg. of cement per square metre) with the result that the modulus of elasticity was improved from 40,000 to 100,000 or even 236,000 kg. per square cm. It is likely that grouting is efficient in hard fissured rock, where the fissures are closed by the grout. Tests in sandstone and marl were attended by little success.

It should be recognized that the results of this last series of tests are of the greatest importance, and it is highly desirable that they should be repeated in all types of rock. The effect of pressure grouting in decreasing the ratio E/E' should be further investigated.

The modulus of elasticity of rock and its compressibility can be obtained by measuring the yield of the rock when subjected directly to a concentrated load, as at Rossens.[36] Alternatively, sections of tunnel have been sealed off and filled with water under controlled pressure. The variation in the tunnel diameter or circumference was then recorded as the pressure on the water was increased. The method has been used at Amsteg, Innertkirchen, Handeck, Rossens, Lucendro, and for several French, Austrian, and Italian tunnels.

Acoustic or seismic methods are also used, the rate of transmission of sound vibrations being related to the degree of fissuring of the rock—the more compact a rock, the higher the wave velocity. This method enabled fissured rock to be detected and avoided during the construction of the inclined shaft of Carla hydro-electric power station near Toulouse.

Recently Gigli in Italy and Remeniéras of Electricité de France have suggested [47, 48] using the waterhammer pressure waves for measuring the overall E value in a tunnel. This method was first proposed in 1934 by the Author,[50] when it was established that the wave velocity depended on the respective values of the modulus of elasticity of the rock, the concrete lining, and the steel shell.

Natural Prestressing of the Rock

It is apparent that the rock inside the mountain is stressed by its own weight before any excavation has been made. It is apparent also that this natural stressing or prestressing of the rock increases with the depth. At the beginning of the twentieth century, the Swiss geologist Heim [13] expanded this theory by adding that, at great depths, the stress at any point must be the same in all directions. This is the hypothesis of the so-called " hydrostatic " stress distribution in the rock. A certain amount of backing was given to these theories by the rock behaviour in deep trans-alpine tunnels (Gothard, Simplon, Lötschberg, etc.) and in deep mines.

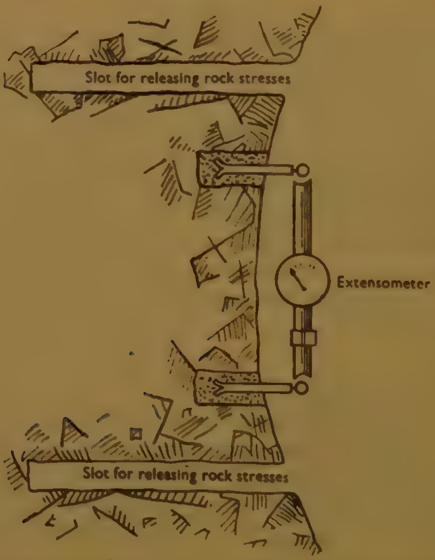
Measurements have recently been made inside deep tunnels and galleries with the result that Heim's theory has been approximately verified, and other theories must be discarded. However, more specific confirmation will be required if the theory is to become more generally accepted by designers.

The measurements will have to be made inside the tunnels as they are being blasted out. If it is to be assumed that, before blasting, the rock pressure is uniformly and hydrostatically distributed, the cutting of the tunnel disturbs this pressure distribution. The pressure in a direction normal to the tunnel periphery is released and the corresponding circumferential pressure near the tunnel, measured along a prolonged horizontal diameter, will be increased—perhaps doubled. So the pressure measured along the tunnel wall may not be the actual rock pressure in the undisturbed state.

If the rock, before blasting, was in plastic equilibrium and completely constrained, it begins to creep when pressures are released by the blasting. This creep may be slow, but there are indications that very brisk displacement may follow the instantaneous release of pressure.

The next difficulty concerns the measuring technique to be used. This is illustrated in *Fig. 9*. An extensometer is fixed to the rock surface. The rock is then bored round the extensometer, which remains on the bore core. A borehole diameter of 235 mm. has been used in France.† As the boring progresses, the pressure round the core is relieved and the readings of the extensometer are recorded. *Fig. 11* shows how, when the depth of the

Fig. 9


Slot for releasing rock stresses
Extensometer
Slot for releasing rock stresses

MEASUREMENT OF ROCK STRAIN INSIDE A TUNNEL (AFTER OBERTI AND TALOBRE) cylindrical boring increases, the strains recorded by the extensometer increase, the ratio between the principal stresses tending towards a constant value, which, for this particular case, is rather higher than unity. The tests to which *Fig.11* refers were made inside the test tunnel of Montpezat. Although, for these measurements at Montpezat, an alternative method was used the results are similar. A horizontal channel is cut out of the rock and a Freyssinet-jack, as used for prestressing concrete, is inserted into the split and grouted into the rock (*Fig. 10*). Extensometers are disposed on the rock surface at convenient angles, so that zero readings correspond to the conditions before cutting out the horizontal channel. After excavation, the pressure inside the jack can be read directly when the readings on the extensometers are readjusted to zero. With this method, the pressure is measured directly, thus eliminating a possible error in calculating the modulus of elasticity E.

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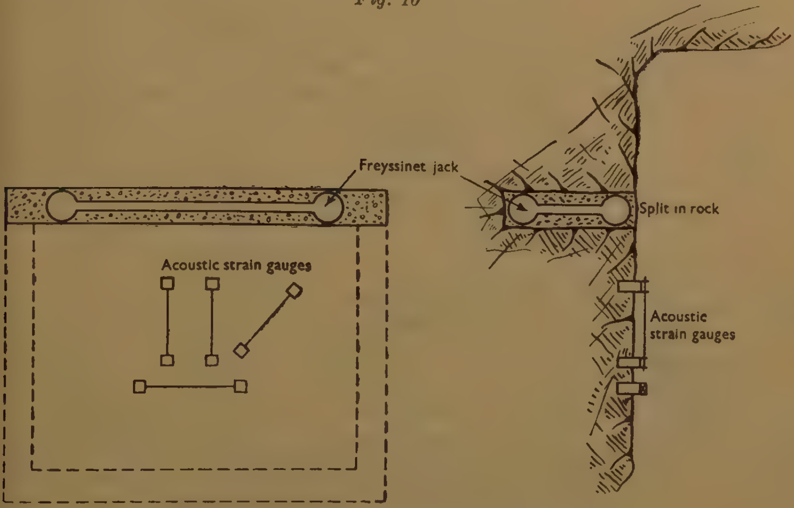
† This method was first used in Italy by Oberti at Venina, where an annular channel of 2 to 3 metres diameter and 0·5 metre depth was excavated round the extensometer [43, 44, 45].

Fig. 10



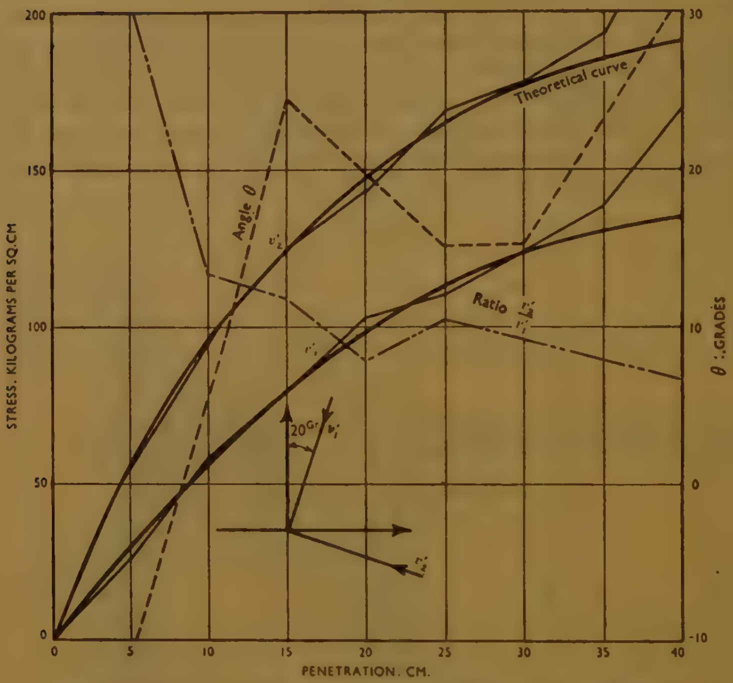
MEASUREMENT OF STRESS INSIDE A TUNNEL

In a third method, the strain meter is arranged for insertion into a borehole of 56 mm. diameter. A co-axial core drill with a diameter of 250 mm. relieves the stresses in the central core. Readings are taken in three directions in order to ascertain the direction of the principal stresses. This method has been used at Isère-Arc [64] but the results have not yet been published.

Behaviour of the Rock Round a Cylindrical Hole or Gallery

In order to interpret correctly the results of rock measurements and for a better understanding of the modern theory of the stability of tunnels and galleries, some further remarks are essential.

Fig. 11



NATURAL PRINCIPAL STRESSES IN GNEISS MEASURED INSIDE THE GALLERY AT MONTPEZAT (AFTER TALOBRE)

It has already been mentioned that drilling a hole or blasting a tunnel profoundly disturbs the pressure distribution in the rock.

Several authors [25, 26, 27, 28, 29, 30, 43, 44, 45, 46] have analysed the behaviour of the rock round a cylindrical tunnel, while there was no internal pressure ($p = 0$) in the tunnel. If the rock pressure p^* produced by the rock's own weight is acting on the circular contour of the tunnel in one direction (vertical) only, the stresses at the tunnel wall on the horizontal axis Oy are, according to Timoshenko : $\sigma_r = 0$; $\sigma_t = 3p^*$ (see *Fig. 12* (*a*)).

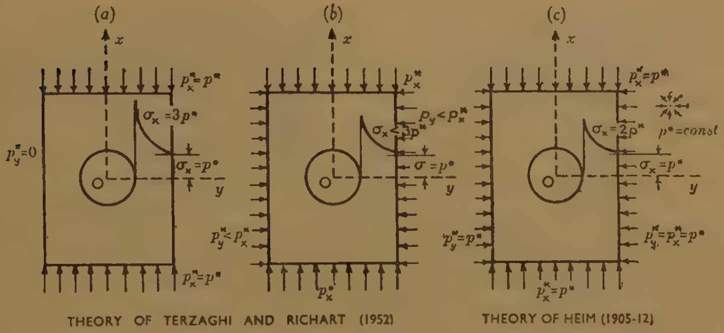
In addition, the theory shows that on a diameter parallel to the pressure p^* (vertical diameter) the stresses are tensile : $\sigma_t = -p^*$. Strains should be observed at the soffit of tunnels if the picture given by the theory were true, and tunnels filled with water at pressure p, causing an additional circumferential stress $-p$, would leak.

The recent theory of Terzaghi and Richart,[19] introducing a component in a direction normal to p^* (due to Poisson's ratio), shows that the stresses are (*Fig. 12* (*b*)) :

$$\sigma_t < 3p^* \text{ on a horizontal diameter.}$$

$$\sigma_t > -p^* \text{ on a vertical diameter.}$$

Fig. 12



THEORY OF TERZAGHI AND RICHART (1952)

THEORY OF HEIM (1905-12)

STRESSES IN THE ROCK AROUND AN EMPTY GALLERY

(*a*) The rock pressure p^* is parallel to direction Ox
(*b*) „ „ „ $p_y < \overset{*}{p_x}$
(*c*) „ „ „ $p^* = \overset{*}{p_x} = \overset{*}{p_y}$ is "hydrostatic"

If the pressure p^* has the character of a hydrostatic pressure ($p_x = p_y = p^*$) (Heim) the stresses on the axis Oy are, as shown in *Fig. 12* (*c*) : $\sigma_r = 0$; $\sigma_t = 2p^*$ for any diameter.

The following remarks are based on the assumption that the theory of Heim is true and that the initial pressure distribution in the rock is of the so-called hydrostatic type, which causes the pressure at the border of the tunnel to be doubled ($\sigma_t = 2p^*$) after excavation of the tunnel. Denoting the limit of elasticity of the rock by σ_{el} and its crushing strength by σ_{cr}, then, if $\sigma_{el} > \sigma_t = 2p^*$, the tunnel is stable. If the local pressure σ_t is greater than the limit of elasticity, plastic deformation of the rock will occur. Experience shows that, although the plastic deformation of the rock is very slow, when once begun it is difficult to stop ; usually it continues until either the plastic deformation has evened out the stress distribution or the rock is completely crushed and the gallery collapses.

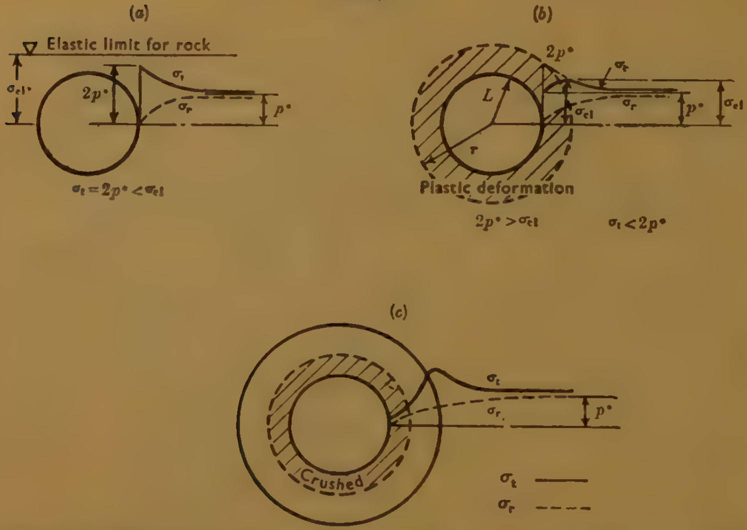
Crushing of the rock will certainly occur if $\sigma_{max} - \sigma_{min} > \sigma_{cr}$. *Fig. 13* (*a*), (*b*), and (*c*) represent, according to Talobre, the probable distribution of pressure for :

$$\sigma_t = 2p^* < \sigma_{el} ; \quad \sigma_t > \sigma_{el} ; \quad \sigma_{max} - \sigma_{min} > \sigma_{cr}$$

respectively. It should be clearly understood that these three sketches give only qualitative information, for they cannot take into account the lack of homogeneity of natural rock.

In the case where the distribution of pressure is determined by the gravitational field alone (*Fig. 12* (*a*)), the stress distribution can be ascertained by model experiments. Tests made for the Krångede tail-race tunnel[57, 58] (Sweden) showed an ogive cross-section to be more favourable than a circular one (*Fig. 14*).

Fig. 13



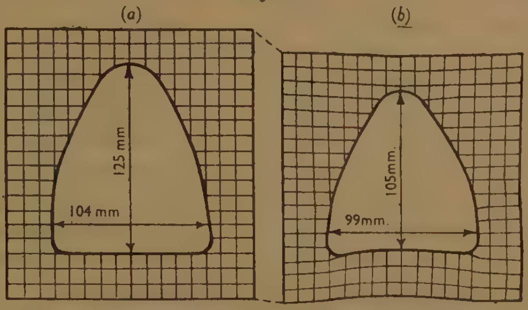
Stresses in the rock round an empty tunnel (after Talobre and others)

Rock Stability adjacent to a Pressure Conduit. Remarks on Steel-Lined Tunnels and Prestressed Pipes. Examples

The hydrostatic pressure inside a conduit, gallery, or tunnel filled with water is $p = wH$ (where H denotes head, w the specific gravity of the water). If $p = p^*$ the conditions round the perimeter of the gallery would be identical to those prevailing before the construction of the tunnel and the rock would revert to the original state of uniform stress distribution p^* = a constant (if the rock is assumed to be homogeneous and the overburden deep). It is easy to check that the stress conditions along the perimeter of the gallery then give $p = \sigma_r = -\sigma_t = p^*$, which, added to the stresses shown in *Fig. 12* (*b*) ($\sigma_r = 0$ and $\sigma_t = 2p^*$), finally yield $\sigma_r = \sigma_t = p^*$ everywhere.

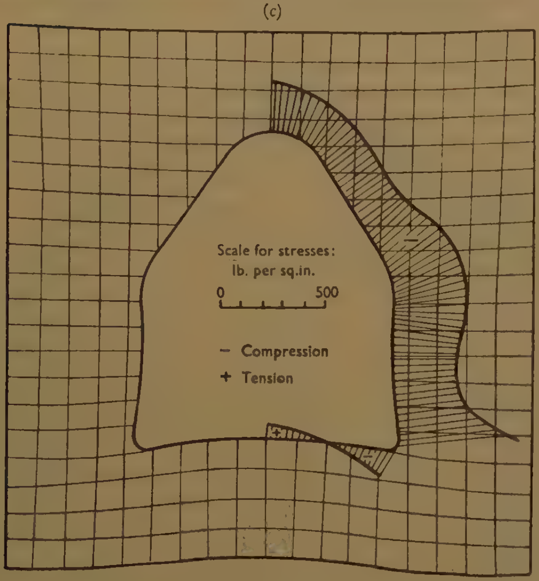
A deep tunnel in sound homogeneous rock with internal water pressure $p < p^*$ would be perfectly safe without any lining, whatever the value of p^*. But rock is rarely sound and homogeneous, and hydro-electric power tunnels and shafts are rarely deep. For these and other reasons the water pressure p should be less than p^* and it is convenient to write $p = p^*/n$, where n has the significance of a safety factor. When using the rule of thumb that the water pressure p inside the tunnel should not be more than half the depth of the overburden ($p = \frac{1}{2}H$), the safety factor n is equal to twice the specific density of the rock, $w = 2{\cdot}5$. This gives $n = 5$, which seems to be a reasonable safety margin to adopt when one considers the safety factors usually applied to steel and concrete. An approximate estimate of the redistribution of the stresses in the rock after introduction of

Fig. 14



BEFORE DEFORMATION

AFTER DEFORMATION



MODEL-TESTS ON KRÅNGEDE TAIL-RACE TUNNEL

the water pressure p in the gallery will be obtained by superposing the stress distribution as shown in *Fig. 3* on that represented in *Fig. 13*. These comments summarize the modern views on pressure shaft theory.

If the prestressing in the rock round a deep gallery is of the "hydrostatic" type, then the pressure σ_t round the gallery perimeter is $\sigma_t = +2p^*$ all round and the tangential stresses produced by the water pressure p will easily be absorbed by the prestressed rock without any resulting tensile stresses being produced. The hydrostatic pressure releases the rock stresses around the tunnel bore.

Conditions are quite different when the water pressure p is greater than the prestressing p^* or when, owing to insufficient overburden, the pressure

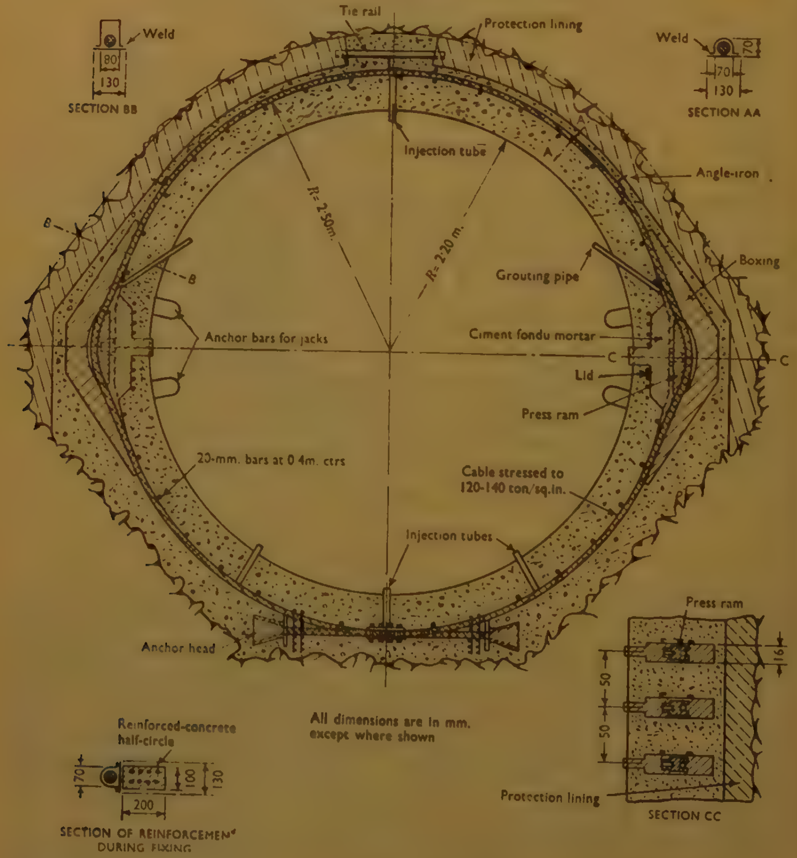
p^* is not of the "hydrostatic" type (that is to say, when there is insufficient horizontal pressure component).

Stress and strain measurements inside tunnels have to be considered as dealing with three-dimensional systems. In several cases, it was thought that model tests were desirable to help interpret the results.

In recent years such basic ideas have led to many new designs, most of them taking advantage of the ability of the rock to absorb pressures. The technique of grouting is of primary importance in the transmission of pressures from the lining to the rock, and is considered in more detail later in the Paper. Other methods can be considered, in which prestressing may be used instead of or in conjunction with grouting.

One of the very first attempts to introduce a radial compression stress σ_c in the concrete lining of a pressure tunnel was made at Marèges (*Fig. 15*).

Fig. 15



PRESTRESSING THE TUNNEL LINING AT MARÈGES (AFTER MARY)

There, Mary[83] incorporated circular cables running outside the concrete lining; these cables were stressed by hydraulic jacks, the stresses in the cables being calculated so that the final resulting circumferential stress σ_c in the lining remained always a compression stress, even when the gallery was filled with water under full pressure.

This is a typical prestressed structure which was devised at a time when the modern methods for prestressing were still in their infancy. With the pressure pipe invented for Marèges, the prestressing method puts all the load on the cables and no advantage is taken of the bulk resistance of the rock. There are other examples of similar constructions.

Kieser has patented a method (to be described later) of forcing grout behind annular rings of precast concrete. An alternative method is to use Freyssinet-type flat jacks placed in the concrete in the direction of generators of the cylindrical tunnel lining. These jacks would produce the circumferential pressure necessary for prestressing the concrete lining.

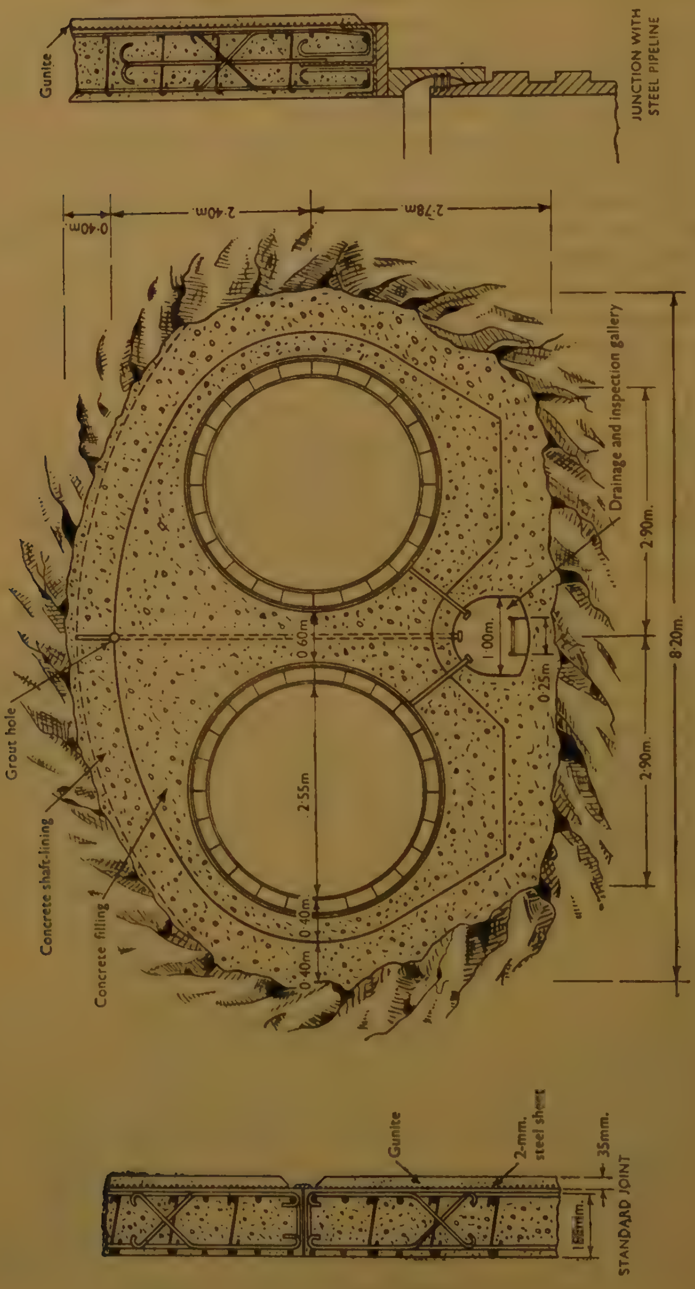
The four prestressed concrete pipelines of Soverzene[45, 73] are placed in pairs in two inclined shafts of elliptical cross-section. Each section of pipe is 4·5 metres (14 feet 9 inches) long, 2·55 metres (8 feet 4½ inches) internal diameter, and 0·19 metre (7½ inches) thick. The sections were precast off the job in a vertical position. High-frequency vibration was used to consolidate the concrete which was poured into the annulus formed between two cylinders of steel plate, the inner of which was collapsible, whilst the outer one, which was 2 mm. thick, remained attached to the concrete and became a permanent watertight lining. After 24 days' seasoning the pipes were armoured around the 2-mm. plate and prestressed with a 5-mm.-diameter high-tensile-steel wire; this was done in a machine designed for the purpose. After completion of the armouring, an outer layer of gunite was applied (*Fig. 16*). The pipes were then placed in the inclined shaft and fitted together by butt-welded joints on the ends of the steel liners. Finally, the space between the pipes and the rock was filled with vibrated concrete.

The 270-mm.-diameter pressure conduit at Muccone operates under 120 metres head. The pipe is in concrete, precast in vacuum, reinforced and prestressed with high-tensile-steel wires. A space of 50 mm. was left round the prestressed pipe for grouting with pressure grout.

Outside Water Pressure and the Danger of Buckling of the Steel Lining

The construction of the steel-lined shaft at Handeck I station clearly illustrates the problems involved. After the pressure shaft had been under full pressure for a short period and then emptied, inspection showed at some places that the steel lining was buckling under the external water pressure. This was attributed to the great length of the pipes being used, the method of concreting and the quality of the concrete filling between the steel plates and the rock. Ground-water penetrating into the voids of the concrete was believed to have caused the outside pressure to rise until buckling of the pipes resulted. Repair was effected by rewelding the damaged pipes

Fig. 16



Typical cross-section through the precast concrete penstock at Soverzene and grouting. Similar buckling was reported at some of the Italian pressure tunnels of the Marinoni type, which are lined with very thin steel plates (see Table 2).

As a result of these experiences, vibration and grouting of the concrete filling was then universally adopted. Nevertheless, inspection of steel-lined tunnels after grouting may reveal areas where voids have not been filled with grout and where buckling may occur.

It should be emphasized that no rock or concrete is completely impervious. Hydrostatic pressure may be transmitted long distances through small rock fissures or through the pores of the concrete. It must, therefore, be assumed that, since the concrete lining of galleries and shafts is somewhere in direct contact with water (for example, with the water in the surge tank), full hydrostatic pressure may be operating behind the steel lining.

In earlier projects efforts were made to relieve this pressure by drainage pipes parallel to the tunnel. If these pipes are small, running just beneath the steel lining, they probably become sealed when pressure grouting is applied. If they are large, designed as inspection galleries, experience has proved that the stress distribution on the circular tunnel lining is no longer symmetrical. Bending moments will develop in the lining, which may cause a disastrous burst, such as occurred in the Gerlos pressure shaft in Austria. Modern views of designers are against any drainage system with pipes or galleries parallel to the main pressure tunnel or shaft.

If small drainage pipes are provided, they should not be considered as relieving the water pressure in the rock, even though some of them keep working and spouting clear water years after the rock has been grouted solid.

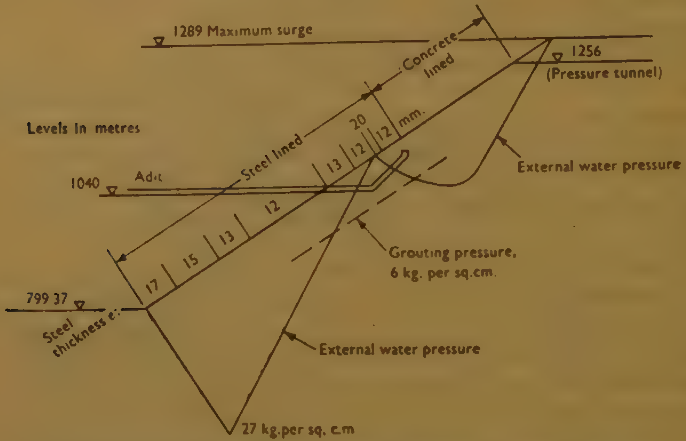
Local springs and wet rock surfaces should be conveniently drained; after final grouting of the rock, the drainage should be grouted solid with thick grout.

On several recent projects, such as Kemano, Gondo, and some Norwegian schemes, the inclined pressure shaft is divided into two sections by an adit which is used during the construction. This adit is then kept open as a permanent drainage gallery and provided with drainage boreholes so that the external pressure on the lining may be reduced. *Fig. 17* shows the drainage system and the distribution of external pressure which was considered for the design of the steel-lined high-pressure gallery at Gondo.

At the present time, most designers agree that the external water pressure is a major problem, but opinions diverge considerably regarding the distribution of the external pressure and even more so regarding the methods for stiffening the steel lining and the design of the lining against buckling.[38, 39]

Some designers claim that any reinforcing rib or anchorage of the steel lining would make it impossible to ensure a perfect concrete filling behind the plate linings, so they suppress all forms of anchorage. A typical modern example is the pressure shaft of the Gondo power station where the pipes are not anchored in the concrete and had to be reinforced internally during the concreting process to withstand the pressure of the wet concrete. The shaft was finally grouted at low pressure (*Fig. 17*).

Fig. 17



GONDO PRESSURE SHAFT. DIAGRAM OF EXTERNAL WATER PRESSURE. DRAINAGE THROUGH ADIT

There are many modern examples of the opposite technique, in which the plate linings are solidly anchored to the concrete. It is a matter of detailed design to shape the anchorages so that they do not hamper the dense packing of the concrete lining. The grout holes need to be so arranged that grout pressure will penetrate to every point where voids might have been formed during the initial concreting.

Table 3 gives some details of the reinforcing hoops or ribs which were adopted at Isère-Arc, Oberaar, and elsewhere. Proposals were made to replace the conventional angle-irons by other types of hoop, or by arranging the plates differently (Isère-Arc). Electricité de France are now introducing the so-called "hedgehog spine" anchorage, which consists of studs welded to the plate; one of the first examples is the steel lining of the Rossens intake and discharge tunnels,[35] where the studs were welded by hand. Electricité de France proposes to introduce stud-welding machines to lower the cost of anchoring.

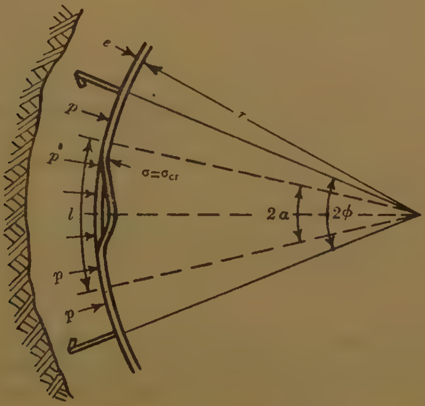
It is well worth noting that ideas concerning the anchorage of steel intake pipes passing through gravity or gravity arch dams were evolved on similar lines. Earlier pipes were not anchored at all. The Rossens intake tunnel is, to some extent, a replica of the anchorages designed by the Compagnie Nationale du Rhône for the pipelines of Génissiat, and the welded studs (hedgehog spine) were first used at Castelo do Bode (Portugal) for the three pipes through the gravity arch dam.

The design calculations for the steel plate linings depend on the form of the anchorage, if any. An inquiry among designers responsible for steel linings shows that this question is still very controversial. The theories of Issemann-Pilarski (which are to some extent a generalization of the theory of Allievi for free pipes) and the formulae of Von Mises, Föppl, and Amstutz[41] (first Paper) have been used. They all apply to the case of a pipe, reinforced by ribs or not reinforced at all, but not directly backed by the concrete filling. The theories of Issemann-Pilarski and Föppl are based on the theory of thin shells and they introduce a critical value p_{cr}' for the external pressure p' at which the lining suddenly becomes unstable and buckles. More recently a detailed Paper by Juillard[42] and a second Paper by Amstutz[41] discuss the effect of prestressing (by grouting) the thin steel shell in order to increase its stability against buckling.

Another approach was given by Dubas when calculating the steel lining of the gallery of Rossens. This method does not introduce the idea of the general instability of a shell but assumes that, when a lining buckles, the metal has somewhere reached the limit of elasticity.

Dubas assumed (*Fig. 18*) that the reinforcement and anchorages are along a generator of the cylinder. If 2ϕ is the angle at the centre corresponding to the angular opening between two successive anchorages, Dubas assumes that the bending stress in the steel plate at point A reaches the yield point, thus producing buckling when the plate is no longer backed by the concrete filling over a length $l = 2\alpha r$, where 2α is the angle at the centre of the buckling length l.

Fig. 18



Calculation of local stresses and strains when a steel lining is buckling (after Dubas)

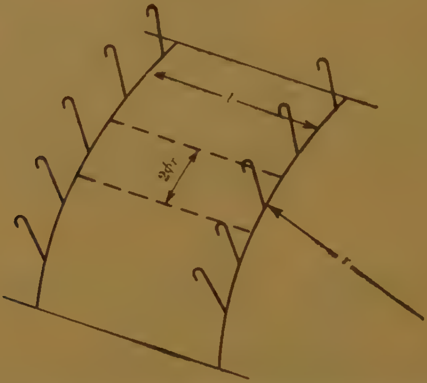
He finds that $\alpha^3 = \frac{6pr}{eE}\phi$ and

$$\sigma_{cr} = \frac{2\alpha^2 r^2}{e^2} p_{cr}' = \sigma_{el} \text{ (yield point)}$$

from which the critical load p_{cr}' can be calculated. This method was used for calculating the steel lining of the shaft of Handeck II power station.[66]

These ideas have been further elaborated in an unpublished internal report of Electricité de France, where it is shown that the anchorages or ribs are better arranged along circles of the cylinder (*Fig. 19*).

Fig. 19



SYSTEM OF ANCHORAGE PROPOSED BY SOME FRENCH SPECIALISTS

With this arrangement the loads acting on the struts can be calculated accurately.

It will be found that the weight of steel being added to the steel plates for these " hedgehog spine " anchorages is very small.

Grouting

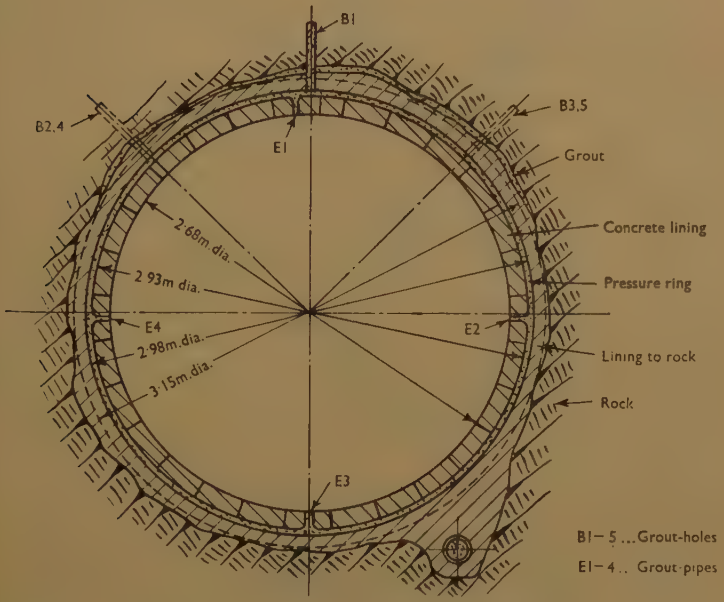
The importance of grouting for redistributing the stresses in the rock and for applying compression to the concrete lining and to the steel shell, thus creating radial and tangential stresses, has been realized only very recently. The effect of grouting on the rock elasticity has been shown in recent years (*Figs 21* and *22*).

The obvious danger of inadequate concrete backing to the lining should be countered by systematic grouting between the steel lining and the concrete, the grouting holes being concentrated along the steel reinforcement ribs and between the concrete and rock. It is also suggested that grouting increases the stability of the steel lining against buckling.

The technique of grouting was developed with the intention of suppressing water leakage through the rock when no steel lining was provided, as well as to stop the ground-water pressure penetrating to the steel shell.

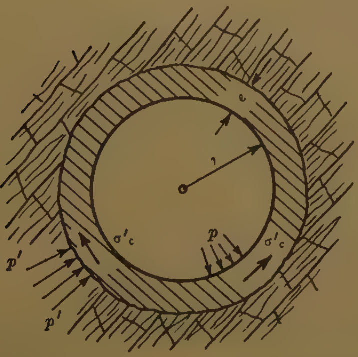
Systematic rock grouting was used in 1940 for the 10-km.-long pressure tunnel at Innertkirchen,[53, 54] in order to prestress, in compression, the thin precast curved asbestos-cement plates with which the tunnel is lined. This is probably the first example of prestressing being used for a tunnel on such a large scale.

Fig. 20



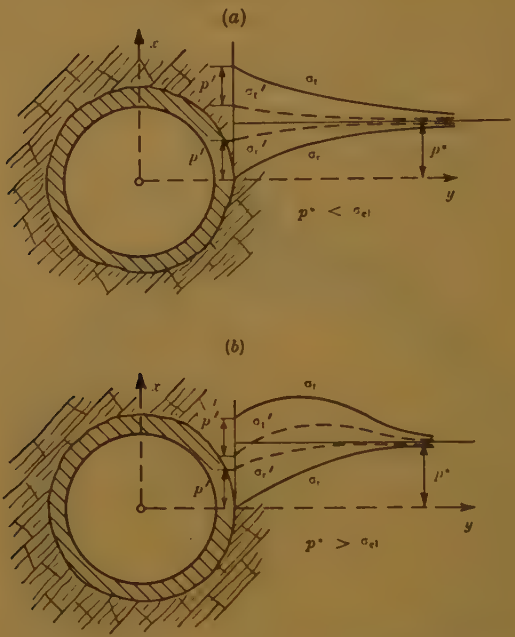
GROUTED PRESTRESSED GALLERY (AFTER KIESER)

Fig. 21



PRESSURE GROUTING BEHIND A CONCRETE LINING IN UNSOUND ROCK

Figs 22



PRESSURE DISTRIBUTION BEHIND A CONCRETE LINING AFTER GROUTING

(*a*) Sound Rock : $p^* < \sigma_{el}$
(*b*) Plastic rock : $p^* > \sigma_{el}$

p^* denotes prestress pressure
p' ,, grout pressure
σ_{el} ,, yield point of the rock
σ_t, σ_r denote stresses before grouting
σ'_t, σ'_r ,, ,, after grouting

Since then several designers have proposed ways to facilitate grouting procedure. Kieser [32, 33] in Austria has patented procedures in which precast concrete blocks replace the ordinary concrete lining. They are masoned against a rough concrete lining covering the rock surface. The precast blocks are provided with a small circular groove running along the outside of the concrete lining (*Fig. 20*). Grout is forced in these grooves, compressing the cylinder formed by the precast blocks. The consequent prestressing should be greater than the stresses created by the water pressure p inside the pressure tunnel.

It is believed that Swiss engineers at Mauvoisin, Switzerland, contemplate the deliberate provision of free space between the concrete ring and the rock by the use of a special net of expanded metal. At intervals the expanded metal net ends in a concrete seal which closes the air space. The proposal allows for five or six grout holes in a tunnel length of 8 metres, the grout pressure being 50 per cent higher than the water pressure in the tunnel. In addition, the French are considering the use of high-pressure grouting for steel-lined shafts using, instead of ordinary pipes, banded pipes or the new French pipes prestressed with cables. They claim that the latter are not likely to buckle so long as the cable remains in tension.

The pressure to be used for grouting is still a controversial point.

The grouting of galleries in unsound or broken rock with no cohesion constitutes a special problem. With no cohesion and with zero stress round the perimeter of the gallery, equilibrium cannot exist and the rock would collapse under its own weight. A radial compressive stress must therefore be applied if equilibrium is to be maintained. This stress can be applied by supporting ribs, by bolts, or by a concrete lining perhaps in conjunction with grouting. The magnitude of this compressive stress is given by the Mohr stress diagram. Assuming that the angle of limiting friction in rock can be taken as 45 degress for all practical purposes, the Mohr circle shows that this stress would be equal to one-sixth or one-seventh of p^*, where p^* denotes the measured compression in rock (assumed to be $p^* = 2{\cdot}5H$).

Thus, to stabilize fissured rock, a radial compression equal to about one-seventh the measured compression p^* has to be applied to it. This is usually done by grouting behind the lining to a minimum pressure of $p^*/7$ to $p^*/6$.

If a water pressure p has to be supported by a tunnel lining of concrete only (no steel), then the grouting pressure has to be raised to a higher value if fissures and seepage are to be avoided. An internal water pressure p produces tensile stresses (*Fig. 21*) of $\sigma_c = \dfrac{pr}{e}$ in the concrete lining ring, where r denotes the radius of the lining and e the thickness of the lining.

A grouting pressure p' produces in this concrete lining a compression stress : $\sigma_c' = p'r/e$.

It is known that, owing to the creep of cement, the pressure falls after the grout has set and it is also a fact that, during injection, the pressure is much reduced between the grout holes. For these reasons, it is usually assumed that the compression induced in the concrete ring is lessened by at least a half. On this, the usual factor of safety is applied to ensure that the value of σ_c' is not exceeded by the applied water pressure. This factor of safety is usually taken as 2 so that the relation giving the grouting pressure should be $p' = 4p$.

On the other hand, it is desirable that the grouting pressure should not exceed $\frac{1}{2}p^* = 1{\cdot}25H$. This finally gives :

$$p = \frac{1{\cdot}25}{4}H \simeq 0{\cdot}3H$$

It will be seen that these recommended values of p and p' will not be practical for all cases.

It is emphasized that the grouting operation should be carried out with considerable care and that a thorough grouting of the lining is necessary to ensure an even distribution of the pressure.

In many cases this value $p = 0{\cdot}3H$ can be increased, because the concrete can be relied upon to withstand some tension (up to the maximum of 20 kg. per square cm.), or because it may be decided that p' can rise above the value $\frac{1}{2}p^*$, or, alternatively, because the concrete has been reinforced. The latter method tends to be very expensive depending upon the relative proportion of steel to concrete. The steel reinforcement has to be designed for low working stresses (of the order of 5,000 lb. per square inch). A gunite lining on steel mesh can yield to a certain extent and higher stresses are allowed.

The assumption that the rock cohesion is nil will usually be pessimistic and furthermore the rock quality will improve with grouting of the cracks and the safety of the surrounding rock is thus improved. All these factors give an additional safety to the concrete lining and may permit a raising of the internal water pressure p; however, with little or no precise information available on this subject, it is not possible to suggest a numerical value.

Some values of the grout pressure p' can be given for the practice with sound rock. It will be seen that they do not agree with the recommended value of $p' = 4p$. At Gondo, where the maximum internal water pressure is 18 kg. per square cm., the pressure shaft was grouted as follows :—

	Kg. per sq. cm.
Steel-lined shaft below level 1085 :	
Grouting between concrete and steel . . .	6
Grouting in fissures	12
Concrete-lined shaft above level 1085 :	
Grouting on sections without steel linings .	25

A second series of grout holes were driven into the rock and grouted at a pressure of 6 kg. per square cm. to compress the concrete ring against the steel ring and to eliminate any voids between the steel lining and the concrete. This method proved to be successful.

For grouting behind steel linings, some engineers of Electricité de France propose the following procedure :—

(1) Low-pressure (5 kg. per square cm.) grout behind concrete for filling voids.

(2) 15 days later, re-drill the grout holes into the rock and grout to a pressure of $p' = 1\frac{1}{2}$ times p, the water pressure in the tunnel.

(3) 15 days later still, low-pressure (5 kg.) grout behind the steel lining at low pressure to fill voids.

Some Swiss and Italian engineers seem to agree with high-pressure grouting at $p' = 1{\cdot}5p$. German engineers propose $p' = 2p$ and Bernard[17] mentions grout pressures of 30 kg. per square cm. in tunnels in Morocco. At Mauvoisin the grouting pressure is 40 to 60 kg. per square cm. for the foundations of the dam.

A rule of thumb commonly used assumes

$$p = \tfrac{1}{2}H = p^*/5 \text{ which yields } p' = 1{\cdot}5p = 0{\cdot}75H = 0{\cdot}3p^*$$

At Lovero (Italy), two injections with $p' = 5$ to $10p$ have been used to make watertight the tunnel built in crystalline schists. Special arrangements were made to avoid the buckling of the steel lining.

It is to be expected that the technique for grouting pressure tunnels will develop in the near future towards higher values of the grout pressure.

Rules for the Design of Pressure Tunnels and Lined Shafts

This discussion should aim at giving some rules for the design of linings of pressure galleries. At this stage it can be done only very tentatively. Talobre,[27] who has issued an internal memorandum to Electricité de France on this problem, concluded by saying that the safety of any tunnel depends on the care taken on the spot during the construction and grouting.

The grouting pressure p' behind the concrete lining should prestress it to such an extent that no fissure can develop when the water pressure p is applied. The grouting pressure behind the steel lining should make sure that no large voids remain behind the steel, even when the tunnel is emptied at lowest temperature. A reasonable stiffening of the steel plates against buckling should be produced. In many cases the grouting pressure p' will exceed the external water pressure and the steel linings will be designed accordingly.

The strains and stresses in rock, concrete, and steel resulting from grouting should be estimated.

Testing of Pressure Galleries: Water Seepage

Most lined pressure galleries are tested after construction. When the construction of the steel-lined pressure shaft at Oberaar (Grimsel) was complete, it was filled with water and subjected to a pressure test, with a maximum pressure of 40 atmospheres at the middle of the shaft. This corresponded to 66·8 atmospheres at the base of the shaft, which was equivalent to working pressure plus allowance for waterhammer at the base of the shaft. The volume of the water introduced for a given rise in pressure was noted and hence the stress in the steel lining was deduced. It was found to be 483 kg. per square cm., corresponding to a pressure of 40 atmospheres and about 800 kg. per square cm. for 65 atmospheres. It was also computed that the steel lining was carrying a quarter of the total load and the surrounding rock three-quarters. It is also planned that the pressure shaft will be tested periodically for watertightness, and the intervals between tests will range from 6 months to possibly 3 years if results are satisfactory.

The inclined pressure gallery of Gondo is in two parts; the lower part below level 1085 is steel-lined, the upper part between level 1085 and 1289 (next surge level) with a maximum head of 204 metres is concrete-lined.

All the steel pipes to be used as lining for the lower part were tested in the works.

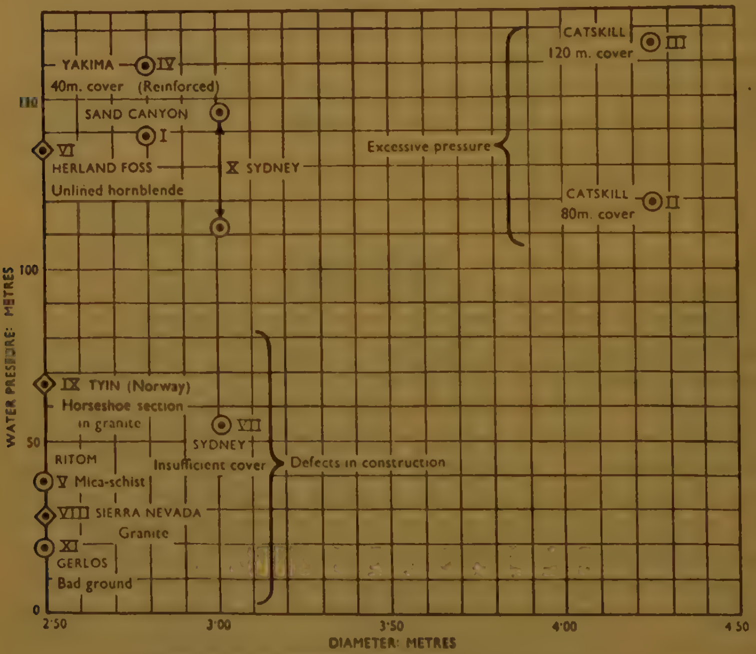
During the final pressure tests on the completed shaft, a water loss of 10 litres per minute was measured for the total length, including the unlined section. For the steel-lined section only the loss was 5·7 litres per hour.

Talobre has referred to the possibility of testing the watertightness of small drill holes in order to get an idea of the water losses in the whole tunnel. It is approximately true to say that the losses depend upon the tunnel length rather than upon its diameter.

Accidents and Failures in Pressure Tunnels

In recent years enquiries have been made into the nature of ruptures in pressure conduits, pipelines, and tunnels caused by water hammer.[51] More recently, Talobre published a diagram,[25] which is reproduced in *Fig. 23*, analysing the ruptures of several steel-lined pressure galleries resulting from lack of rock resistance. This diagram does not include the rupture which occurred in the thin steel lining of the Mese pressure shaft, which was caused by internal pressure, nor the ruptures due to external water pressure at several places.

Fig. 23



Rupture of pressure galleries (after Talobre)

Cracks in the steel lining parallel to the axis of the shaft occur mostly near a crack in the concrete lining or a fault in the rock.

Talobre [25] mentions the rupture of a steel lining 6 mm. thick, which was able to withstand the full internal water pressure without reaching the yield point. The rupture occurred along a rock fissure where the concrete lining had been weakened by a spring. It is assumed that local concentrations of pressure and the resulting bending stresses in the pipe were the cause of the failure.

The most serious accident recorded in recent times was the failure of the pressure shaft at Gerlos (Austria). The rock consists of schists, mostly of doubtful quality, and the overburden is meagre. Ruptures occurred at four points (marked I to IV on *Fig. 24*) in October 1945 and September 1948. Although the pipe was designed to withstand the full hydrostatic pressure without reaching the yield point, it was, in all four cases, cracked along lines parallel to the drainage gallery. A very considerable quantity of rock was washed away and the generating sets were flooded to the point of immersion. It is believed that the rupture was produced by uneven subsidence of the concrete packing near the drainage gallery, which developed bending stresses in the steel lining (*Fig. 25*). This example emphasizes the importance of a dense concrete filling behind the steel lining.

Rupture of the lining may also occur near an underground power house at the transit from sound cohesive rock to a fissured and unreliable rock formation which is nearer to the surface. This is an important argument in favour of locating the power house well inside the mountain in the same sound rock formation as the shaft itself.

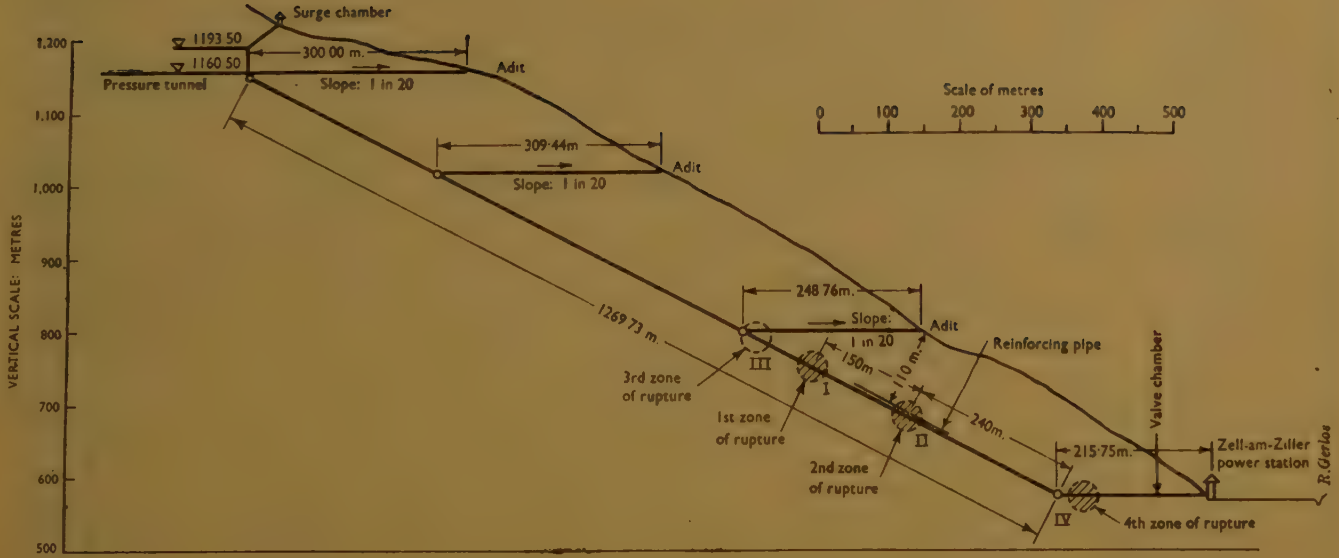
It has been related how a large fissure in the rock was bridged over at Gondo Power Station. At Santa Giustina,[72, 74] the pressure conduit is in hard fissured limestone whilst the power house is in plastic sandstone. Instead of a steel-lined pressure shaft it was decided to design an independent and self-supporting welded pipeline (3·7 to 3·5 metres in diameter) inside an inclined gallery, 6·0 by 4·85 metres, leaving enough room for inspecting the pipe (*Fig. 26*).

Very little information has been published relating to the ruptures of ordinary lined tunnels and shafts resulting from failure of the rock by internal water pressure. The Ritom tunnel constitutes one of the very few cases where the results of the enquiry have been published.[22]

DATA ON LARGE PRESSURE SHAFTS

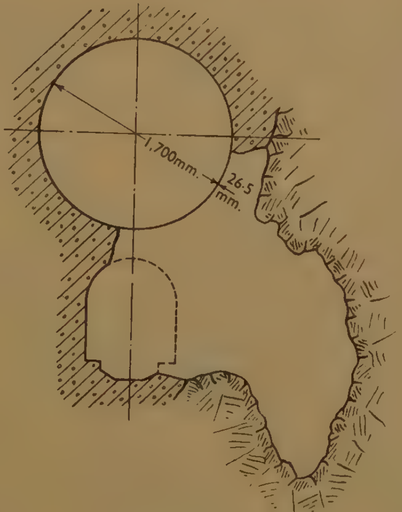
Data on large pressure shafts have been summarized in Tables 1, 5, and 6. Table 5, part of which is from a publication by Talobre, gives general data on existing large steel-lined shafts. Table 1 compares the calculated stresses in the steel linings at the base of the shafts, assuming that no load is transmitted to the rock. These calculations are based on

Fig. 24



THE PRESSURE SHAFT AT GERLOS, AUSTRIA

Fig. 25



Rupture No. 2 at Gerlos

TABLE 5.—DATA ON LARGE STEEL-LINED PRESSURE SHAFTS

Name	Country	Date of commission	Gross head : m.	Internal dia. : m.	Rock	No. of pipes	Discharge : oumecs	Gradient : tan α
Lavey .	Switz.	1950	42·5	5·8	—	3	200	—
Porjus .	Sweden	1914	57	3·5–42	Granite	7 + 2	90	Vert.
Niagara .	U.S.A.	1920	60	4·7	Limestone	1	—	Vert.
Krångede	Sweden	1936	60	5·5	—	4	—	Vert.
Niva .	U.S.S.R.	1949	74	4·5	—	4	4 × 62·5	Vert.
Castillon	France	1949	90	3·4	Good limestone	2	72	0·35
Cordéac .	France	1946	92·5	2·9–3·8	Limestone	1 + 1	29 + 28	0·71–0·02
Châtelot	France	—	91	3·0	Limestone	1	40	0·702
Piage-de-Vizille .	France	1950	142·4	3·5	—	1	45	0·549
Isère-Arc	France	1953	153	2–3·7	Gneiss, mica-schists	2	2 × 50	Vert.
Salarno .	Italy	1945	162	1·6–2·0	—	1	3	Vert.
Boulder .	U.S.A.	1936	177	9·14–7·62 3·96–2·59	—	—	—	1·00 0·70
Valabres	France	1954	195	2·2	Limestone	1	15	0·85
Sillre .	Sweden	1923	197	1·85	—	1	—	0·19
Soverzene	Italy	1948	285/225	2·55	—	2	2 × 20	3·00
Brevières	France	1951	232	3·75	Quartzites	1	50	0·02
Cedegolo	Italy	Constr.	234	2·60	—	—	—	Vert.
Rossaaga	Norway	Constr.	245	3·3–2·8	—	1	34	1·00
Brommat	France	1933	260	4·0–3·0 –2·6	Granite	3	—	Vert.

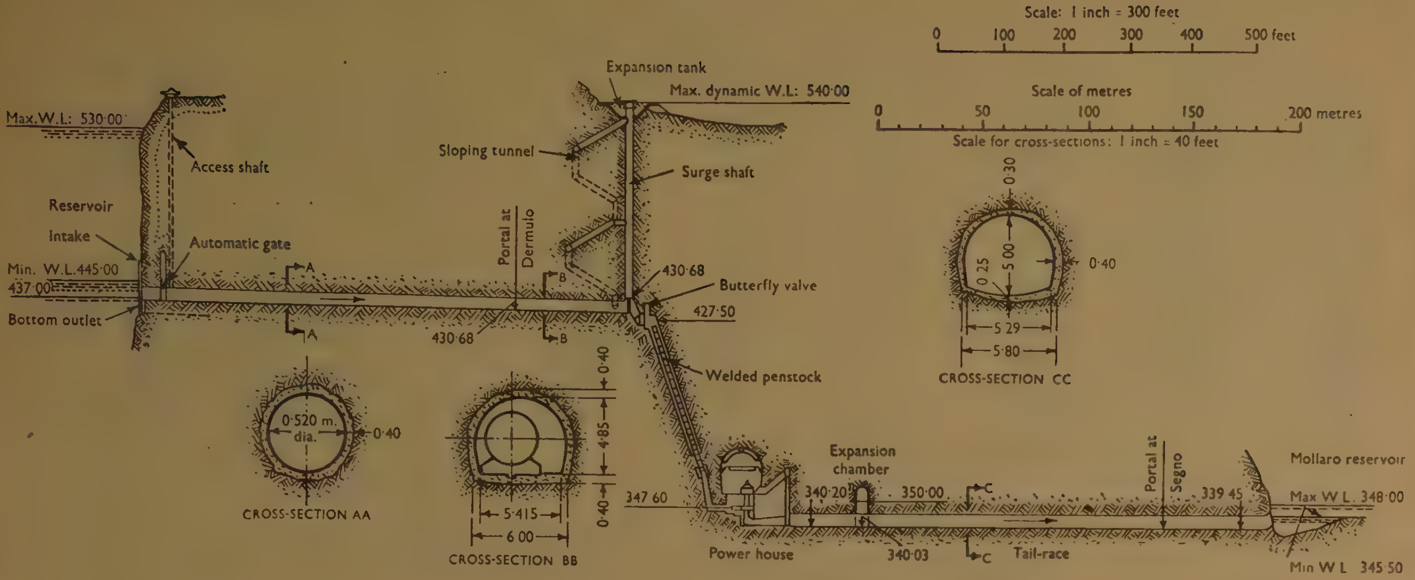
TABLE 5.—*Continued*

Name	Country	Date of commission	Gross head : m.	Internal diam. : m.	Rock	No. of pipes	Discharge : cumecs	Gradient : tan α
Bersimis	Canada	Constr.	267	9·45 3·06–2·36	— —	1 8	416 52	Horiz. 1·19–0·01
Wassen .	Switz.	1949	281	2·4–2·2	—	2	2 × 42	0·67–0·013
Castelbello .	Italy	1949	294	3·25–2·80	—	1	21	0·85
Verbano	Switz.	1953	297	3·0–2·80	—	1	44	—
Mera I .	Italy	1948	330	2·55–2·25	—	1	—	0·70
Snowy Mountain T 1	Australia	Constr.	336	3·65	—	2	2 × 64	Vert.
Rodund III .	Austria	1943	339	3·2–3·05	—	1	60	0·58–0·0
Tafjord II	Norway	Constr.	396	1·75	—	1	60	0·93
Achensee	Austria	1927	406	2·3	Limestone (Dol.)	1	—	1·00
Cañon del Pato .	Peru	Constr.	420	1·8	Granite	1	28	Vert.
Mauvoisin-Fionnay	Switz.	Constr.	459	2·4	Calcar. schists	1	23	—
Handeck II . .	Switz.	1950	463	2·25–2·15	Granite	1	30	0·61–0·15
Pallazeno	Italy	1926	527	1·8–1·5	Gneiss (mediocre)	1	8·4	0·70
Oberaar .	Switz.	1953	531	1·65	—	1	—	—
Handeck I	Switz.	1932	546	2·3–2·1	—	1	8	0·72–0·08
Santa Massenza	Italy	1949	600	2·6–2·5	Limestone (excellent)	1	20	0·85
Gerlos	Austria	1943	620	2·2–1·6	Limestone (fissured)	2 1	2 × 35 12	0·52
Lyse . .	Norway	Constr.	630	2·0	Granite	—	—	0·70
Montpezat .	France	1953	643·5	2·6	Granite	1	22	0·467
Innertkirchen	Switz.	1940	672	2·6–2·4	Granite and gneiss	1	40	0·616 0·583
Rovesca	Italy	1926	720	1·2	Schists, gneiss	1	3	0·123 0·70
Meso .	Italy	1926	758	1·8	Gneiss (mediocre)	1	—	0·70
Kemano	Canada	Constr.	790	3·35	—	4	4 × 196	1·11
Pfaffensprung	Switz.	Proj.	823	2·5	—	1	4 × 30	0·70–0·60
Grande-Dixence-Fionnay	Switz.	Constr.	872	3·0–2·8	—	1	45	—
Albigna .	Switz.	Proj.	1,046	1·5	—	1	—	—
Roselend	France	Proj.	1,171	3·0	Schists	1	50	0·90

the data available from many publications. Table 6 refers to the maximum power concentrated in single large shafts.

The information contained in these three Tables is self-explanatory and illustrates the scope of modern projects better than any comment.

Fig. 26



The Santa Giustina power station

TABLE 6.—WATER-HORSE-POWER OF LARGE SHAFTS

Year	Power station	Head:		Discharge per shaft:		Power per shaft: h.p.	Total power: h.p.
		m.	ft	cumecs	cusecs		
1942	Innertkirchen	672	2,200	40	1,410	325,000	325,000
1947	Lumiei	480·6	1,580	16·5	583	77,000	77,000
1951	Santa Giustina	183/95	600/311	22·2/15·9	780/562	142,500	142,500
1952	Santa Massenza	590/460	1,940/1,505	70	2,470	200,000	400,000
1952	Handeck II	463	1,515	30	1,060	154,000	154,000
1953	Isère-Arc	152	500	50	1,770	90,000	180,000
1954	Montpezat	634	2,080	22	779	162,000	162,000
1954	Kemano	790	2,597	196	6,920	560,000	2,240,000
Constr.	Bersimis	253	830	416	14,700	1,600,000/ 200,000*	1,600,000
Proj.	Roselend	1,171	3,840	50	1,770	660,000	660,000
,,	Pfaffensprung	820	2,690	30	1,060	245,000	980,000
Constr.	Grande-Dixence-Fionnay	872	2,860	45	1,590	390,000	390,000
,,	Mauvoisin-Fionnay	459	1,400	23	815	170,000	170,000
,,	Snowy Mountain T 1	336	1,105	128	4,530	270,000	540,000

* The main shaft, 570 feet long, divides into eight penstocks.

DESIGN AND CONSTRUCTION OF UNDERGROUND POWER HOUSES

The dimensions of underground power houses depend on the size and number of generating sets and the station arrangement. The turbines may be impulse wheels (Pelton) or reaction wheels (Francis or Kaplan), and of vertical or horizontal type. The basis upon which rests the choice between the different types of turbines is essentially the same as for ordinary above-ground stations, but increased importance is attached to the volume of rock excavation. The vertical Francis turbine often requires less excavation for the valves and distribution pipes.

The transformers are sometimes arranged within the same genera excavation as the generating sets, as at Innertkirchen and Isère-Arc. They can also be located in a separate excavation parallel to the main excavation, as at Vinstra and Soverzene, or nearby, as at Harsprånget. The last choice is an above-ground location, as at Krångede. A simple rule of thumb used by Italian engineers is that, if the length of the low-tension transmission cables is more than 250 to 300 metres, the arrangement with underground transformers is cheaper. Norwegian designers seem to favour placing the transformers underground, partly for military reasons.

The valves upstream of the turbines can be located inside the main excavation or in a special excavation upstream of the general turbine hall. This last arrangement is reserved mainly for high-head schemes.

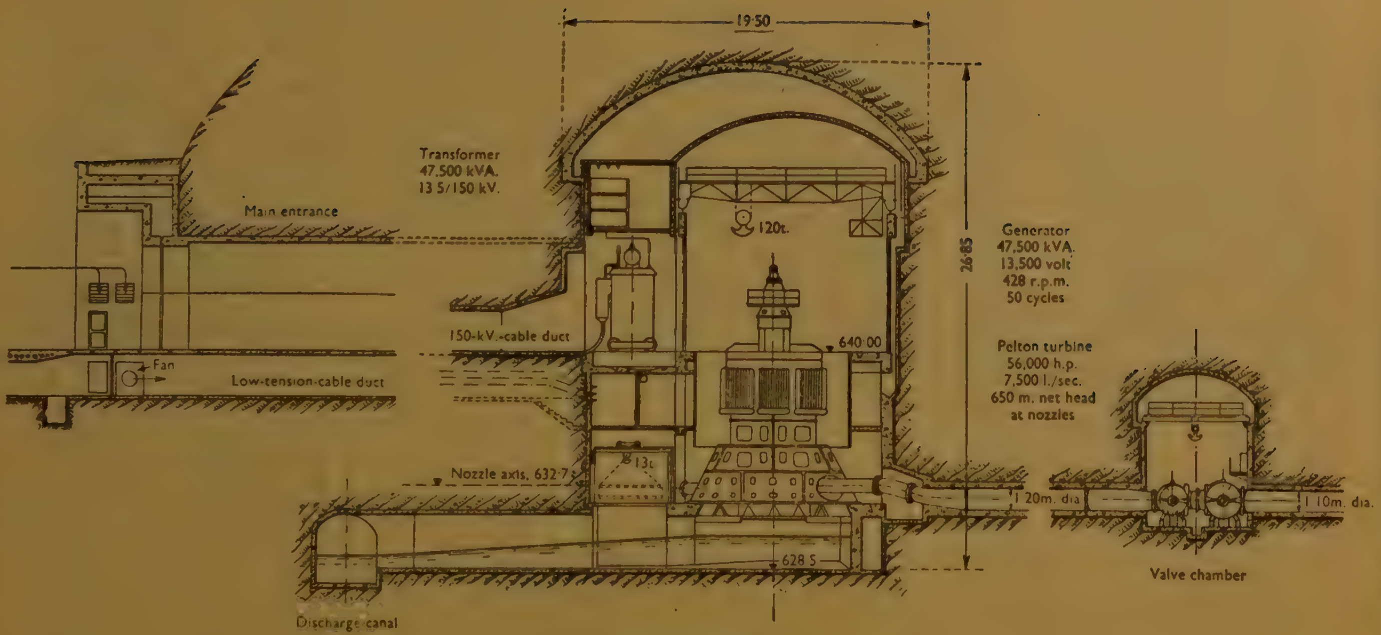
Fig. 27 (*a*) to (*f*) are sketches of the principal designs which have been evolved, and Tables 7 and 8 summarize the main data of several well-known installations.

The technique of excavating the power house will depend on the rock, on its stability and strength, on the main direction of the strata, on the amount of water to be expected, on the cost of excavation, and on the constructional equipment which the contractor is prepared to use on the job. Italian contractors, who have relatively cheap labour available, will adopt techniques differing from those of Swedish or American contractors. But there is a definite tendency, in countries like France, Switzerland, or even Italy, for contractors to adopt Swedish or American plant and methods.

Normally, the excavation and the concreting of the arched roof of the power-house excavation will be given first priority, and then the excavation work will proceed downwards over the whole width of the power house, as at Lavey,[55, 56] or in trenches along the walls, leaving a central core to be excavated later, as at Innertkirchen and Verbano.

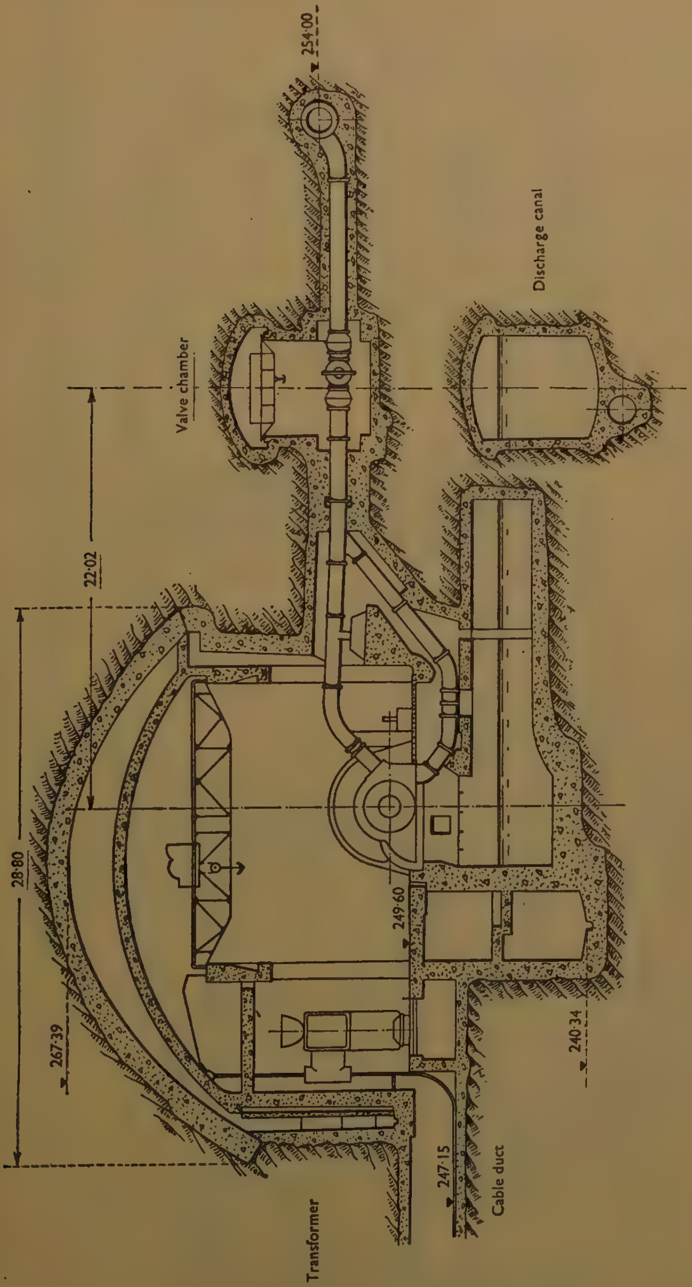
The walls will usually be lined with concrete, the concrete adhering directly to the rock. Norwegian engineers prefer to leave a space of from $\frac{1}{2}$ to 1 metre between the rock and the walls and roof. Thorough ventilation is necessary in order to keep the rock dry. This technique has also been used in countries other than Norway and Sweden (for example, at Lavey [52] in Switzerland). In solid granite the walls may be without any

Fig. 27 (a)



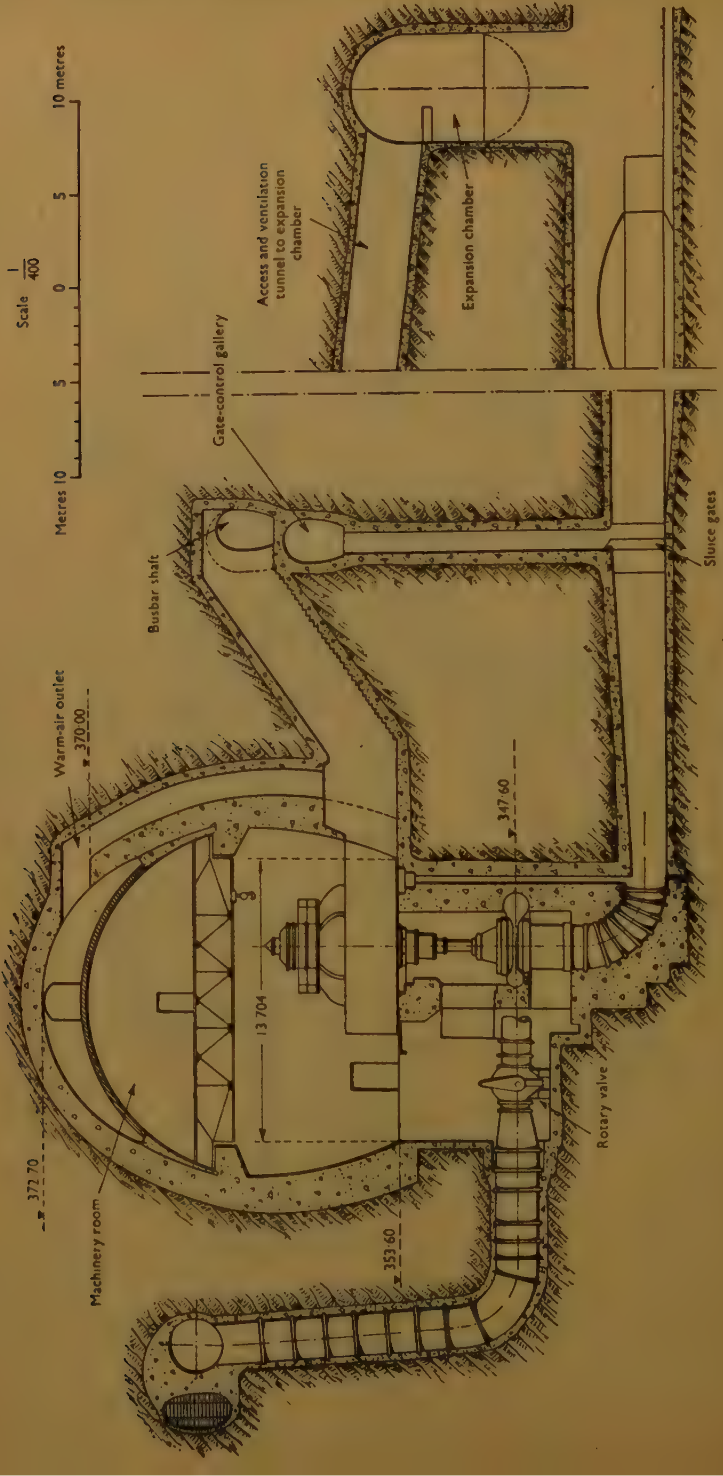
INNERTKIRCHEN
(Gross static head : 672 metres)

Fig. 27 (b)



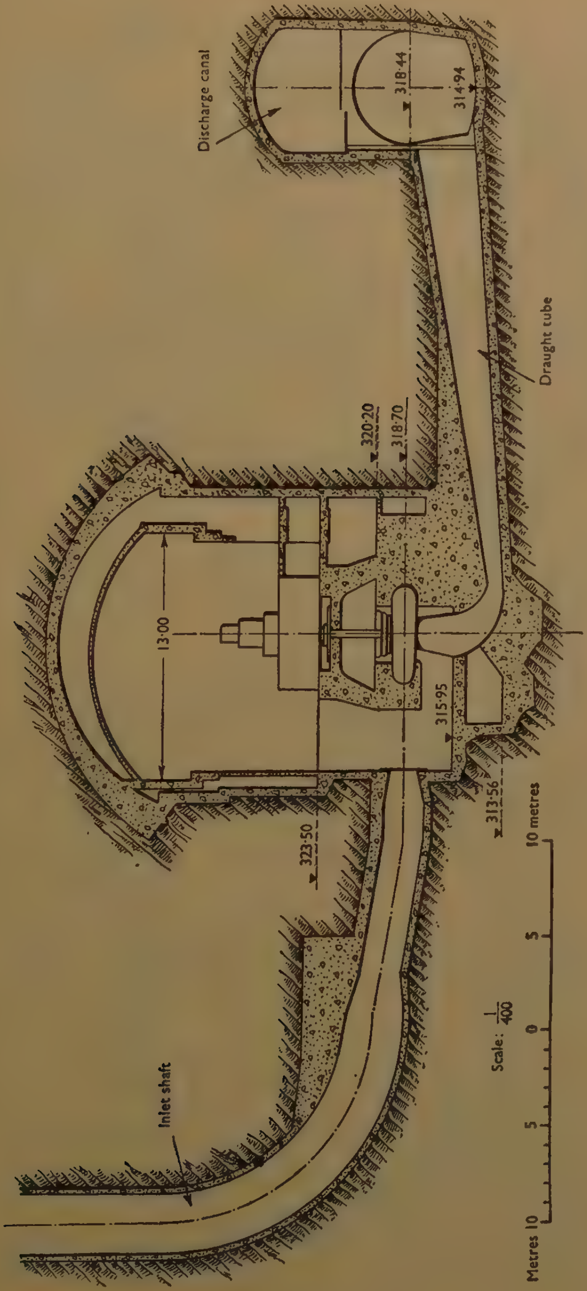
SANTA MASSENZA
(Gross static head : 590/460 metres)

Fig. 27 (c)



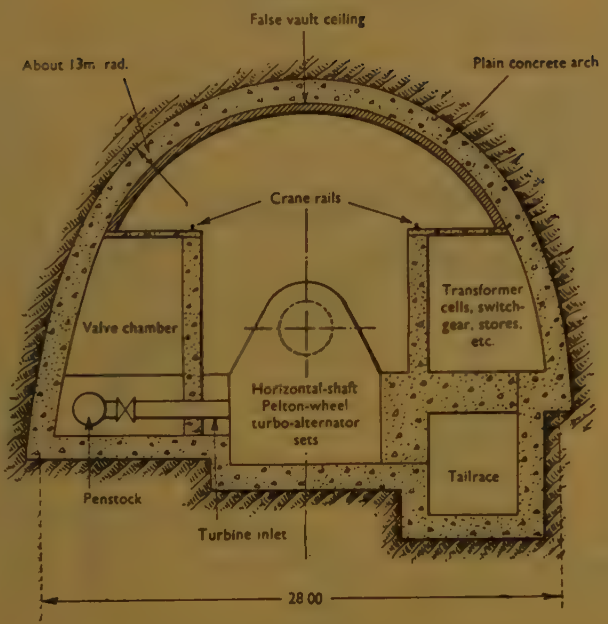
Santa Giustina
(Gross static head : 183/95 metres)

Fig. 27 (d)



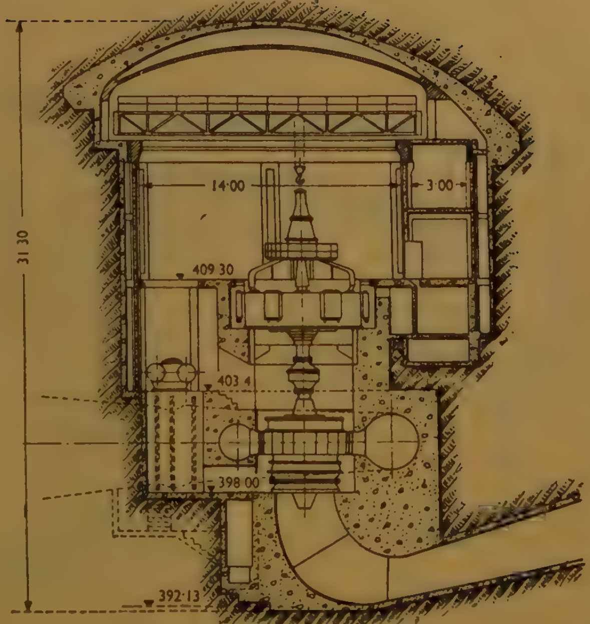
ISÈRE-ARC
(Gross static head : 152 metres)

Fig. 27 (e)



PECCIA
(Gross static head : 426 metres)

Fig. 27 (f)



LAVEY
(Gross static head : 40·6 metres)

Date of commission	Power station	Country	Capacity : kW.	Gross head : ft	Turbines : h. = horizontal v. = vertical	Volume : cu. yds	Excav. for transf. or valve house, etc. : cu. yds	Volume of power house per kW. installed : cu. yds
1910	Porjus	Sweden	60,000	183	8 Francis, h.	35,000	6,000	0·58
1932	Brommat	France	186,000	855	6 Francis, v.	60,000	16,000	0·32
1942	Innertkirchen	Switzerland	210,000	2,200	5 Pelton, v.	59,000	20,000	0·28
1947	Lumiei (Ampezzo)	Italy	58,500	1,465	3 Pelton, h.	33,000	—	0·565
1948	San Giacomo	Italy	200,000	2,140	3 double Pelton, h.	37,000	5,000	0·19
1948	Soverzene	Italy	220,000	870	4 Francis, v.	85,500 (*a*) 102,000 (*b*)	16,500 (*a*)	0·390 (*a*) 0·465 (*b*)
1949	Mar	Norway	180,000	—	—	60,000	—	0·334
?	Vinstra	Norway	180,000		Francis, v.	30,000	8,170	0·167
1949	Provvidenza	Italy	150,000	838	3 Francis, h.	44,000	3,000	0·29
1950	Lavey	Switzerland	76,000	133·5	3 Kaplan, v.	54,500	7,200	0·715
1952	Handeck II	Switzerland	114,000	1,520	4 Pelton, v.	39,000	—	0·34
1951	Santa Giustina	Italy	108,000	560	3 Francis, v.	40,000	—	0·37
1951	Harspranget	Sweden	288,000 (*c*) 380,000 (*d*)	353	3 Francis, v. 4 Francis, v.	63,300 79,000	30,000 (*e*)	0·22 (*c*) 0·206 (*d*)
1951	Ahrzerouftis	Algeria	66,500	1,125	Two 40,000-h.p. Pelton, v. One 7,500-h.p. Francis, v.	44,000 (*f*) 80,000 (*g*)	— —	0·66 1·20
1951	Santa Massenza I Santa Massenza II	Italy Italy	302,000 25,500	1,940 to 1,505	6 Pelton, h. 2 Francis, v.	203,000	10,900 (*h*)	0·92
1952	Monte Argenta	Italy	66,000	164	3 Francis, v.	21,000	8,000	0·32
?	Montorio	Italy	100,000	845	3 Francis, v.	29,000	1,000	0·29
1952	Verbano	Switzerland	100,000	973	4 Francis, v.	33,000	—	0·33
1953	Isère-Arc	France	135,000	500	4 Francis, v.	68,000	—	0·50
1954	Montpezat	France	116,000	2,080	2 double overhang Pelton, v.	26,300	—	0·226
1954	Kemano	Canada	835,000 (*j*) 1,670,000 (*k*)	2,597	8 Pelton, v. 16 Pelton, v.	290,000 473,000	— —	0·347 0·284

Notes. (*a*) Without transformer house.
(*b*) Including transformer house.
(*c*) First stage.
(*d*) Second stage.
(*e*) For transformer house.
(*f*) Useful excavation.
(*g*) Including 10-foot thick walls and roof.
(*h*) For valve house.
(*j*) First stage.
(*k*) Second stage.

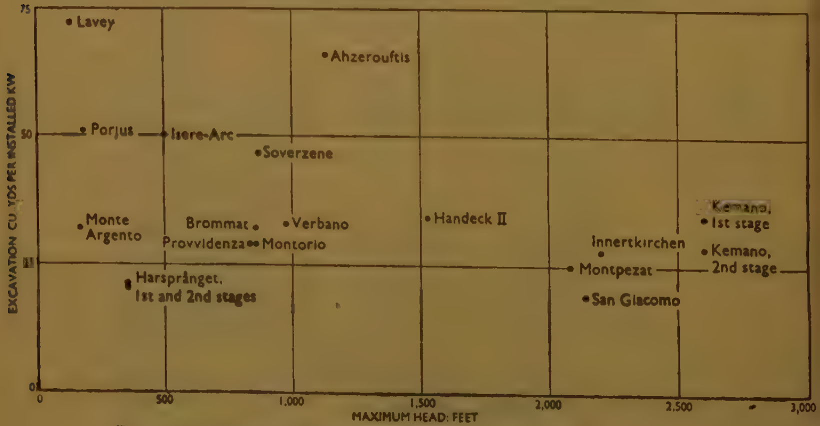
lining and the bare rock is visible, as at Hojum in Sweden. Westerberg mentions one case where the arched concrete roof has been omitted, the rock being covered with expanded metal and a gunited mortar lining. Norwegian engineers favour designs where the concrete arched roof, with or without ribs, is at some distance from the rock roof. They claim that no damage has been caused by falling rock.

A rule of thumb given by Norwegian engineers proposes that the overburden above any power-house excavation should be twice the width of the excavation, with a minimum overburden of 30 to 35 feet. Cases are known (for example, Clachan in Scotland) where the power house was excavated by the cut-and-cover method. At Clachan, this method was cheaper than a conventional power-house design above ground level. For large power stations, where the strata of the rock are suitable, the longitudinal axis of the power house should be at an angle of from 20 to 30 degrees to the axis of the pressure shaft, since this arrangement is favourable for the design of the distribution pipes.

When water is expected along the walls, the wall may be made double with provision for drainage between the skins. If rock pressure is expected to develop, specially reinforced walls are essential for the stability of the excavation.

The walls of the power house at Lumiei are arched in the horizontal direction and comprise segments of cylinders with vertical axes. To deal with higher rock pressures a cylindrical roof and walls of elliptical cross-section are favoured. In the well-known power house of Santa Giustina the elliptical cross-section is reinforced by the heavily reinforced generator floor, which act as a horizontal strut against the horizontal component of the rock pressure.[72, 74] The Peccia and Cavergno power houses (Maggia [69]), with horizontal Pelton wheels are of parabolic shape to withstand the rock pressure (*Fig. 27* (*a*) to (*f*)).

Fig. 28

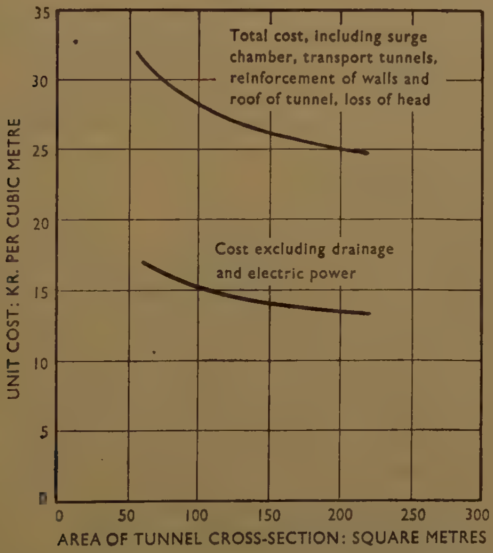


RELATION BETWEEN VOLUME OF EXCAVATION AND GROSS HEAD

The designers of the power house of Ahrzerouftis thought that a semi-circular roof and thick slanting lateral walls would provide additional safety. The French designers estimate that for the worst possible conditions of pressure, the stresses in the arched roof are not more than 65 kg. per square cm. and in the lateral walls 93·8 kg. per square cm.

There is a tendency, when designing underground power houses, to choose the station arrangement giving the minimum excavation. The generator support and the spiral casings are developed along the conventional lines adopted for above-ground stations. Isère-Arc shows one interesting alternative. The conventional cylindrical generator supports for the vertical Francis turbines are replaced by hemi-spherical cupolas, whose diameter is greater than the diameter of the spiral casings. The erection of a turbine was not begun until the civil contractor had built the cupola, so that civil engineering and turbine erection did not interfere with each other.

Fig. 29



COST OF TUNNELS (EXCLUDING ADMINISTRATION) AT 1949 PRICE LEVELS

Vertical access shafts were favoured some years ago but modern contractors prefer longer inclined tunnels which can be traversed by Diesel-engined trucks and heavy lorries.

Mainardis, Heggstad, and others have emphasized the importance of the electro-mechanical equipment required for underground stations, such as the types of generator, cooling system, and aeration to be adopted, and the advantages of having the transformers underground or on the surface.

TABLE 8.—DIMENSIONS OF SOME LARGE UNDERGROUND POWER STATIONS

Power house	Head : metres	Total height*: m.	Total width : m.	Total length : m.	Crane span : m.	Crane capacity : tons	Turbine horse-power	Distance between turbine centres : m.	Position of transformers	Valves
Kemano, 1st stage	790	41	25·5	214	—	2 × 225	8 × 140,000	—	Inside main excavation	Special excavation
Kemano, 2nd stage		41	25·5	348	—	—	16 × 140,000	—	Inside main excavation	Special excavation
Innertkirchen .	672	26·85	19·5	96·70	10·8	120	5 × 56,000	14·50	Inside main excavation	Special excavation (28-ton crane)
Montpezat . .	634	23·2	16·1	50	13·5	2 × 135	2 × 81,000	—	Outside	?
Santa Mazzenza I and II	590/460	28·0	28·8	192·7	14·2	—	4 × 80,000 2 × 40,000 1 × 25,000	18·0	Inside main excavation	Special excavation
Cavergno (Maggia)	590	21·5	28	—	11·6	—	2 × 27,500	—	Inside main excavation	Inside main excavation
Lumiei . . .	480·6	24·2	15·4	71	12·0	—	3 × 25,500	—	Outside	Special excavation upstream of main excavation
Handeck II . .	463	25·55	16·8	79·25	10·70	—	4 × 41,000	14·0	Inside main excavation	Special excavation

Peccia (Maggia) .	426	21·5	28	—	11·6	—	2 × 23,500 2 × 11,600	—	Inside main excavation	Inside main excavation
Ahrzerouftis I and II	345	22·4	20	67	—	120	2 × 40,000 1 × 7,500	—	Outside	
Vinstra . . .	?	27	13	73	11	—	4 × 60,000	11	Inside special excavation	
Bersimis . . .	336	28·5	19·4	171	18·4	2 × 200	8 × 175,000 8 × 200,000	16·8	Inside main excavation	Straight flow valves, inside
Snowy Mountain T 1	334	28	17	92	14·5	—	4 × 134,500	12·8	Inside special excavation	Straight flow valves, inside
Soverzene . .	265	35·1	25·5	73	17	—	4 × 72,500	14	Inside special excavation	Inside main excavation
Brommat . . .	260	31·4	21·8	?	11·5	140	6 × 40,000	—	Outside	Inside main excavation
Santa Giustina .	183/95	29·5	17	61·3	13·7	—	3 × 47,500	12·8	Outside	Inside main excavation
Isère-Arc . . .	152	31·6	16·8	56·25	13	—	4 × 45,000	10	Inside prolonged main excavation	Inside main excavation
Harspranget, 1st stage	108	33 43‡	183	109† 100		2 cranes	4 × 127,000	20	Inside special excavation	—
Lavey . . .	40·6	31	21	64	14·5	2 × 60	3 × 33,500	14	Outside	Separate excavation

* Not including draught-tube height.
† Including transformers and workshop.
‡ Including draught-tube height.

Further details of underground power houses are summarized in Tables 7 and 8, and in *Figs 26* to *29*.

ACKNOWLEDGEMENTS

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The Paper, which was received on the 18th September, 1953, is accompanied by numerous drawings and diagrams from which the Figures in the text have been prepared.

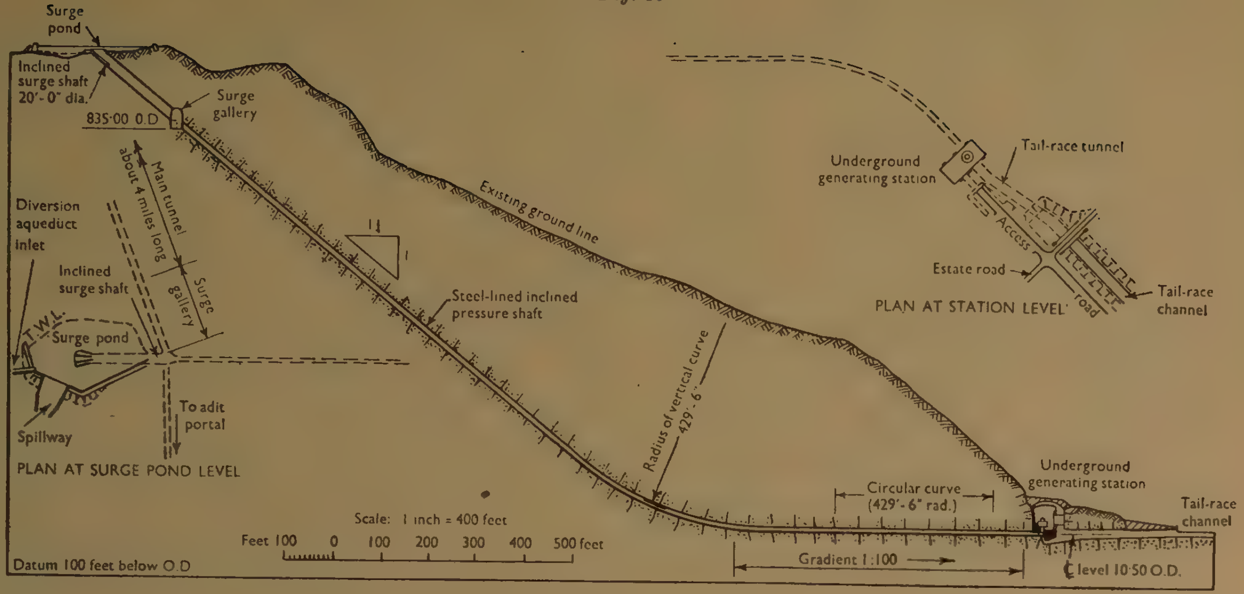
Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr J. A. Banks said that the Author had presented information of great value to all engineers who were concerned with the design of pressure tunnels and similar works. The Paper was particularly welcome at the present time, because it was only in very recent years that the construction of underground generating stations had been undertaken in Great Britain. The Author had referred to the Clachan station in Argyllshire and, whilst others had been constructed since or were in hand, it might be relevant to give some particulars of the Clachan works which formed part of the North of Scotland Hydro-Electric Board's Glen Shira project.

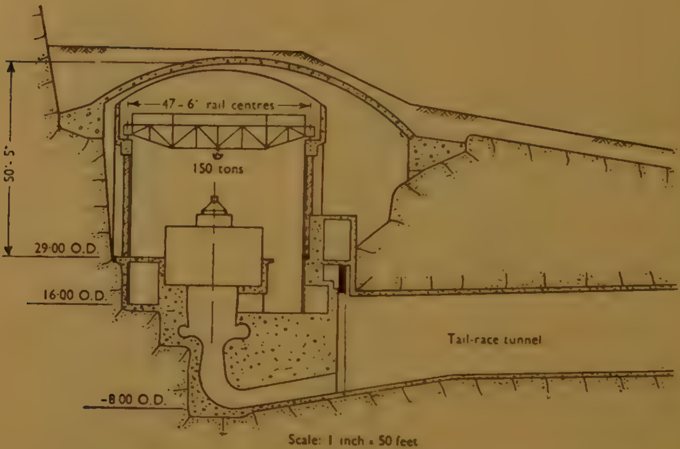
Clachan generating station, with an inclined pressure shaft (the general arrangement of which was shown in *Figs 30*, *31*, and *32*) housed one vertical Francis turbine of 54,000 horse-power operating under a head of 960 feet. As stated in the Paper, the station had been excavated by the cut-and-cover method. The concrete walls below main floor level had been poured hard against the rock and the reinforced concrete columns and crane girders, which carried a 150-ton overhead travelling crane had been anchored into the rock both longitudinally and transversely. There was an inner lining of brickwork and a suspended ceiling, the main roof having been formed by a reinforced concrete arch springing from skewbacks cut into the rock, and with that internal lining and ceiling there was a complete clearance

Fig. 30



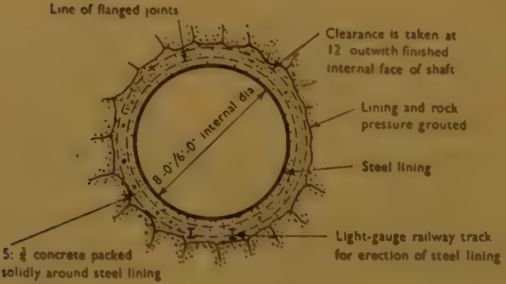
General arrangement, Clachan

Fig. 31



Cross-section through Clachan generating station

Fig. 32


Line of flanged joints
Clearance is taken at 12 outwith finished internal face of shaft
Lining and rock pressure grouted
Steel lining
Light-gauge railway track for erection of steel lining
5: ¾ concrete packed solidly around steel lining
8·0"/6·0" internal dia

Typical cross-section through inclined pressure shaft at Clachan

cavity between the internal structure and the rock itself above the main floor level.

Coming to the pressure shaft, the rock in the power station and through which the pressure shaft had been driven was quartz-schist, and the inclined shaft had been driven at an angle of 39 degrees to the horizontal (1¼ horizontal to 1 vertical). That slope had been adopted for a variety of reasons, but one was that the inclination was just a little in excess of the natural angle of repose of rock spoil, so that the spoil was more easily removed. It so happened that that inclination was almost normal to the planes of cleavage in the rock, so that by driving normal to the planes the whole length of the tunnel had in fact been successfully completed without any support at all, a very distinct advantage.

Excluding the approximately level section approaching the generating station, the depth of the pressure shaft below ground provided a weight of overburden measured vertically above the diameter of the shaft sufficient to balance the water pressure at any point. The lining was of mild steel to British Standard 15 (ultimate strength 28/33 tons per square foot). The basis of the design had been as follows. It had been taken as a first approximation that the stress in the steel lining should not exceed the yield point of the steel, assuming that there was no assistance from cover or other support from the rock itself. Consideration had then been given to the effect of fissures of varying depth in the rock surrounding the encasing concrete (and Mr Banks acknowledged the benefit which the engineers had derived from discussions with the Author on that particular subject), and the results had indicated that it was conservative to design on the basis of yield point.

The lining had also been considered for the effect of external pressure with the shaft empty. It seemed to him that analyses on that aspect were largely empirical, and he was particularly interested in the Author's description of the hedgehog spines as a form of anchorage which had been used in France. From the practical point of view, he was not satisfied that he liked those hedgehog spines. He thought it was advisable to keep down the clearance between the rock and the lining so that the thickness of the casing was reduced and thereby the shrinkage also was reduced. The additional spines would mean greater clearance. Further, from the practical point of view it was not easy to place a lining in an inclined shaft encumbered by spines sticking out of it.

The slides which the Author had shown in illustrating failures in lining made one think twice, but he wondered whether some addition to the normal thickness of the lining and attention to thorough packing and grouting behind the lining was not the direction in which most attention should be given. That, in fact, had been the procedure adopted at Clachan, and perhaps for that reason, being a little conservative, the saving in steel had not been quite so great as the Author indicated as applicable to the general case. Actually, there had been a saving of 350 tons of steel in the Clachan pressure shaft; there were about 700 tons of steel in the shaft and, had there been a surface penstock, about 1,050 tons would have been required.

The thickness of lining ranged from $\frac{7}{16}$ inch at the surge gallery to $1\frac{9}{16}$ inch at the generating station. The diameter was generally about 7 feet 6 inches, but increased to 8 feet towards the surge gallery and reduced to 6 feet at the power station. The reason for the reduction was governed by the maximum thickness of plates both for bending and for welding. The joints were flange joints where otherwise overhead welding would have been involved and a change-over to welded joints was made part-way up the inclined shaft arbitrarily where the static head was of the order of 600 feet. At the power station itself the lining had been designed for the full static head of 960 feet plus allowance for waterhammer.

He was in full agreement with the Author about the importance of thorough packing and grouting. Concrete casing of 1 : 2 : 3 nominal mix with ¾-inch aggregate had been placed by pump as each two pipe lengths of the lining (48 feet) had been completed. The strength of the lining against external pressure had been investigated for a four- and six-lobe collapse, and grouting, which was now proceeding, had been specified to range in pressure from 25 to 200 lb. per square inch. according to the thickness of the lining and the depth of rock cover.

Mr J. Guthrie Brown said that he was interested particularly in that part of the Paper which dealt with the design of pressure tunnels. The Author had brought out very clearly the difficulties in their construction and had detailed the inevitable elements of risk. If a very thin steel lining was used with a view to achieving maximum economy, the main problem was then to prevent buckling by external water pressure. The adoption of steel anchors or hedgehogs, as the Author had described them, to control that buckling was not entirely satisfactory ; he agreed with Mr Banks that their presence interfered very considerably with the placing of the concrete backing.

In his opinion, one of the best means of dealing with the deposition of concrete around steel pipes was to modify the cross-section of the rock to be excavated from roughly circular to pear-shaped. The additional space which that provided above the top of the steel pipeline gave better access for men during the placing and ramming of the concrete.

One of the best types of steel lining, which the Author had referred to but which he thought had not yet been used in Great Britain, was probably the longitudinal corrugated type which, if the concrete or the rock yielded, allowed the steel to extend without undue stress. The Author had referred to it on p. 124 as being designed by Marinoni.

Mr Guthrie Brown was at present engaged on the construction of a concrete high-pressure tunnel without any steel lining, in the North of Scotland. It had a diameter of 15 feet and would operate under a head of 600 feet. When the tunnel had been designed some years previously, there had been an acute shortage of steel in Great Britain and it had been essential that special consideration should be given to economy in steel. Instead of the conventional pipeline on the surface of the hillside, it had been decided to adopt a horizontal high-pressure tunnel. The tunnel dropped 250 feet vertically from the upper tunnel at the surge-shaft, and then ran approximately horizontally for 1,500 feet towards the power station, where, as the rock cover gradually reduced, it trifurcated into three steel-lined tunnels connecting to the machines in the power station. Fortunately, the schist rock through which the tunnel had been driven was of very good massive quality, and confirmed the decision to construct the 1,500-foot length of tunnel without steel lining.

The grouting programme had been divided into three stages. The first (in his opinion by far the most important) was the low-pressure grouting to fill the voids between the concrete lining and the rock. For the primary grouting at 10-foot centres along the tunnel length, the grouting holes were at quarter-points, and the holes were drilled at least 5 feet into the rock; the grouting pressure at that stage was 100 lb. per square inch. The secondary grouting was also carried out at 10-foot centres, and staggered quarter-points, the holes being taken 10 feet into the rocks, with grouting pressures up to 200 lb. per square inch. Finally, the rock was drilled to a depth of 15 feet and grouting at 375 lb. per square inch was adopted. With a static head of 600 feet (260 lb. per square inch), that represented a grouting pressure of about one-and-a-half times the static head. The tunnel was therefore prestressed against the internal pressure to that extent. Finally, holes were drilled at intervals and water tests carried out so as to ensure that all the voids had been filled. Those grouting pressures seemed to agree fairly well with what the Author had indicated.

It was proposed to fill the tunnel very gradually and halt the filling for a short time at about half the working head. At that stage it would be possible very easily to check accurately from the water level in the vertical shaft if there was any slight leakage in the tunnel. In any event, after that it was intended to empty the tunnel and make a final inspection before filling the tunnel to its full head.

It was also intended, before filling the tunnel to paint the interior with several coats of thick bitumen paint, which would serve two useful purposes: it would seal off any small pinholes in the concrete surface, and it would protect the concrete against attack from the peaty water.

Mr W. H. Ward said that during the past 3 years the Building Research Station had been working in collaboration with the Metropolitan Water Board on the behaviour of a water pressure-tunnel in London Clay, which had no steel lining. Although London Clay was very different in character from the types of rock that the Author had discussed, Mr Ward had found it very encouraging to discover that his trends of thought about the London Clay tunnel were very similar to those which were presented in the Paper.

He did not propose to discuss the results of the observations in detail, since they were to be presented to the Institution at a later meeting and would be published during the summer of 1955, but a brief summary of the results might be of interest.

The water tunnel, which had been constructed by the Metropolitan Water Board in London, was about 95 feet deep, and he understood that the ideas behind the initial design had been, first, that the full overburden pressure would develop in the lining, and secondly, that it was reasonable to assume that the London Clay was adequately watertight. Obviously, it was not too difficult to accept what was called in the Paper Heim's theory of an equal all-round pressure, equivalent to the overburden, on an empty tunnel in London Clay. There was a certain amount of evidence that that was the case, and at the present time he had under observation several hundred strain gauges in empty tunnels in London, which were intended to supplement that evidence. It therefore seemed quite logical to argue that it was not necessary to use a watertight lining for a pressure head of water less than the overburden pressure. So the engineers had proceeded with the construction of a lining consisting of unreinforced concrete segments which were jacked into position by means of the shield. The construction was similar to that described by Mr Scott,[85] but the steel lining had been omitted. When construction was complete it had been necessary to find out if the concrete lining, which could not take any tension, would collapse when the tunnel was filled with water. Observations had shown that the circumferential thrust and the radial pressures between the lining and the clay built up as the segments were jacked into position, and that subsequently practically no change in load took place while the tunnel was empty for about 1 year. It seemed to him that no earth pressure, in the usual sense of the word, had developed on the lining. In fact, if he adopted the Author's expression, the concrete segments and the clay had been "prestressed" by the thrust from the shield, and that prestress had not altered since the day it had been applied. The average compression strength of the London Clay was about 300 lb. per square inch, which was quite high compared with the full overburden stress of about 80 lb. per square inch. He thought that was the reason why no stress had developed in the lining other than the stress which had been induced in it as a result of jacking from the shield. When the tunnel had been filled with water to a moderate head—moderate compared to the overburden pressure—the circumferential thrust and the stress between the lining and the clay had dropped off almost in proportion to the applied water pressure. Then, when the water had been emptied out, the stress in the lining had returned to about the same value as it had been before. That process had been repeated and for short-term loading it appeared that the lining and the clay had behaved approximately as an elastic system with the full water pressure acting on the clay boundary. The water pressure had not been sufficient to relieve the " prestress " in the lining completely, and there was no risk of collapse.

The Author had mentioned the use of jacks to prestress concrete linings, and he thought some undesirable features of that system which had been noted in the London pressure tunnel might be avoided in future. When jacking at one point in a concrete ring backing on to London Clay he had found that the friction could be so large that there might be no prestress at a point diametrically opposite the jack. The gradient in the prestress might be very marked if the concrete backed on to hard rock and that should be avoided by adopting methods which reduced the friction.

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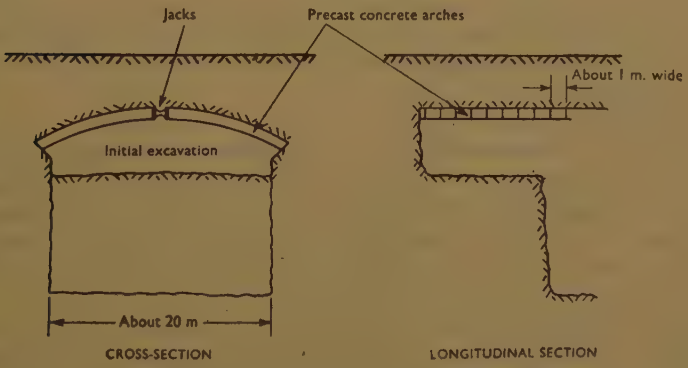
[85] P. A. Scott, " A 75-inch-diameter Water Main in Tunnel: a New Method of Tunnelling in London Clay." Proc. Instn Civ. Engrs, Pt I, vol. 1, p. 302 (May 1952).

The problem was solved by the use of grout for prestressing, but it was not evident how much control there was over the degree of prestress in that method. An alternative would be to use jacks together with some plastic medium between the concrete and the rock, which reduced the friction to a negligible amount, and the Metropolitan Water Board had made some useful investigations into that technique.

Like other speakers, Mr Ward did not favour the use of anchorages between a steel lining and the concrete in order to avoid collapse under the external water pressure when a tunnel was emptied ; it seemed to him that the anchorages were not satisfactory unless there was an almost continuous ring of concrete. He would prefer to see more insistence placed upon compact cavity-free grouting and concreting at the back of the steel lining.

While in Stockholm in the summer of 1954, Mr Ward had seen the construction of a new underground railway tunnel in solid gneiss. One of the stations was comparable in size with the power station described in the Paper. The rock cover was quite thin, being a few metres thick, and a large building was later to be erected on top. The situation was rather similar to that described by Mr Banks at Clachan. In part of the station a system of jacked concrete arches was being used to support the roof and to avoid disturbance of the surface rock (see *Fig. 33*). Two concrete units formed the roof span of about 20 metres. Mr Ward thought that that form of construction was preferable to open excavation in a built-up area, since no great nuisance was caused by blasting.

Fig. 33



SKETCH OF JACKED CONCRETE ARCHES FOR THE ROOF OF AN UNDERGROUND RAILWAY STATION IN STOCKHOLM

Mr A. C. Meigh, referring to the brief indication of tunnelling costs given at the end of the Paper, said that where tunnels had to be constructed through unsound rock the cost of temporary support could be an appreciable proportion of the total cost. The method of support known as roof bolting could effect a considerable saving. Roof bolting had been used extensively in American mines for a number of years, and more recently in British mines, both as an adjunct to conventional support and as a separate method of roof suspension, and it had been adopted to a limited extent in American civil engineering work.

He wished to outline roof-bolting methods and to describe briefly the use of roof bolting in the recent construction in Scotland of a large-diameter tunnel for the housing of electrical apparatus. It was, so far as he knew, the first use of roof bolting in British civil engineering practice.

Fig. 34 illustrated the method. Holes were drilled radially into the roof. The hole diameter was 1¼ inch for a 1-inch-diameter bolt, which was the usual size. The simplest and most effective method of anchorage was the slot-and-wedge method shown in the Figure. A slot about 6 inches long was flame-cut in the end of the bolt and the bolt was driven over the wedge using a pneumatic drill. A nut was placed on the end to support a plate, or alternatively a strip, over a series of bolts, and the nut was tightened to give a tension of about 10,000 lb. In that particular tunnel the bolts had been placed on the top heading, which was widened out from a pilot heading.

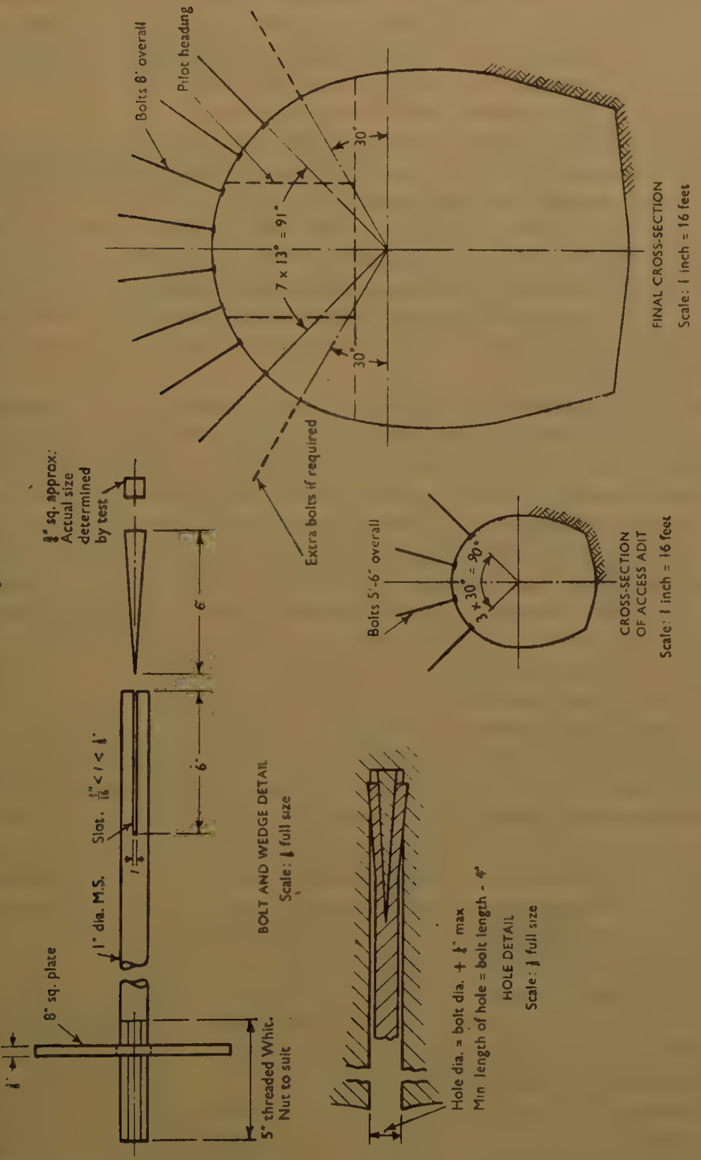
No adequate theory of roof bolting existed, but its general effect could be described in two ways: first, that of preventing progressive yield or creep of the rock through opening of the fissures, and secondly, that of securing the immediate roof to the zone less affected by the excavation. To be fully effective, it was essential that the bolts should be placed right up to the face as soon as possible after firing. That took advantage of the half-dome support provided by the face and the walls of the tunnel.

Judgement was required in selecting the bolt length for a given set of conditions. To aid that judgement, Mr Meigh proposed a semi-empirical formula: $L = K(K_1a + K_2)$ in which K_1a denoted the height of the tension zone above the tunnel crown as given for homogeneous rock by the theory of Terzaghi and Richart, a denoted half the tunnel width, K_2 a constant length to provide anchorage beyond the tension zone, and K a factor which attempted to allow for the nature of the rock. At present it was rather difficult to assign values to that factor, but it could be done by classifying the rock rather on the lines suggested by Terzaghi in his empirical rules for the design of steel sets.

It was not always possible in practice to position the bolts strictly in accordance with the proposed layout. The placing of the bolts had to be modified to suit the rock conditions encountered.

The advantages claimed for roof bolting were, first, that it was cheaper to install than the conventional roof supports; and secondly, when used

Fig. 34



ROOF-BOLTING DETAILS

as a complete method of support, it left an unobstructed surface with no lagging wedges or loose material to remove before placing the permanent lining. The difficulties of compacting concrete adjacent to steel sets were avoided. Furthermore, it was thought that the increased tightness of the roof which resulted from the use of roof bolting should reduce the amount of grouting required around pressure tunnels.

Roof bolting had a limited range of application, and Mr Meigh did not think that it could be used in some of the tunnels described by the Author in his introduction. The Author had, he thought, presented evidence for the theory of Terzaghi and Richart, in that they allowed a range of ratios between vertical and horizontal stresses. The Heim theory stipulated a ratio of 1; Terzaghi and Richart specified a range of stress from 0 to about 2. Mr Meigh thought that at least one of the slides shown by the Author, when introducing the Paper, gave evidence for a horizontal stress greater than the vertical stress.

Mr C. M. Roberts referred to pp. 117 and 118 of the Paper where the Author gave some reasons for building power stations underground and a classification of such power stations under three types. For the purpose of his remarks Mr Roberts suggested another classification in which there were only two categories, namely, those stations placed underground because of the topography or geology at the site of the station (in that category he would place stations with short tail-race tunnels, such as many in Norway, Italy, and Switzerland), and those stations placed underground because of the design concept of the project as a whole. In the latter category were stations with low or medium heads and long tail-race tunnels, such as were found in Sweden.

On the River Moriston in the North of Scotland two underground stations were at present in course of construction, and he wished to describe them briefly, for they clearly illustrated the difference between the two categories which he had mentioned. He would also like to give the reasons for building the stations underground.

The developments in question formed two steps on the River Moriston, the upper one incorporating the Ceannacroc power station and the lower one the Glenmoriston station.

The particulars, which could be added to the information given in Table 7, were as follows:—

	Ceannacroc	Glenmoriston
Date of commission	1955	1957
Capacity: kilowatts . . .	20,000	32,000
Gross head: feet	299	311
Turbines	1 × 21,440 h.p. Francis, V 1 × 5,360 h.p. Francis, V	2 × 21,440 h.p. Francis, V
Volume: cubic yards . . .	9,800	12,700

	Ceannacroc	Glenmoriston
Excavation for transformer or valve house	—	—
Vol. of power house per kW. installed : cubic yards . .	0·49	0·40

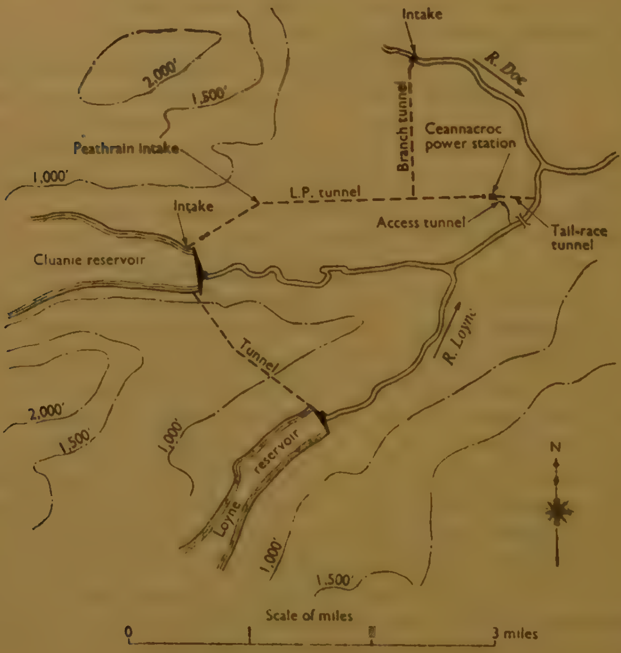
The volume of power-house excavation per kilowatt installed was higher at Ceannacroc because the small 5,360-horse-power machine required almost as much excavation as the larger machine.

Mr Roberts explained that both Upper and Lower Moriston projects had originally been designed as tail developments with surface stations and described the steps which had led to the change of the design to underground stations. First, a study of the head and loading characteristics had shown the possibility of standardizing the three large machines and designing them as three identical units of 16 megawatts thus saving manufacturing time and cost. Attention had then been turned to the layouts of the projects. Taking first Lower Moriston (although Upper Moriston was in fact being constructed first) the surface station had been abandoned for the following reasons. Not only was there a need to economize in the use of steel, as Mr Guthrie Brown had mentioned, but there had also been, at the time of designing the Glenmoriston station, an urgent need to save cement ; by moving the power station to the upstream position it had been possible to arrange for the 4½ miles of tunnel to be unlined except where weak rock required lining. There had thus been a great saving in cement. With modern tunnelling methods it was really much quicker to drive the tunnel in that way and omit the lining, although it meant increased excavation. It required only a very short approach pipe to the power station and therefore saved steel. The turbine being near the intake meant that there was no pressure-relief valve on the turbine and a smaller flywheel effect was required for the alternator. One amenity reason was that it saved a long high-tension transmission line in Lower Glen Moriston. For all those reasons, the Glenmoriston station had been placed underground at about 300 feet below the surface as a head development—which illustrated the second category that Mr Roberts had mentioned at the beginning of his remarks. Having arrived at an underground-power-station arrangement for Lower Moriston, it followed that, because of the standardization of the machines, Ceannacroc station of the Upper Moriston project also had to be underground in a similar position relative to the surge shaft as the Glenmoriston station held towards the head-pond.

Figs 35 and *36* showed the Upper Moriston works, which fell in the first category mentioned above. The point to note was the relative position of the main reservoir, power tunnel, and Doe branch tunnel which fixed the position of the power station at the downstream end of the tunnel with a short free-flowing tail-race tunnel. The power station was close to the surge chamber and connected to it by a high-pressure shaft lined with concrete and grouted, and by short steel-lined tunnels. The tail-race tunnel was unlined except on the invert.

Figs 37 and *38* showed the Lower Moriston works which fell in the second category. The arrangement there comprised a headpond, intake, concrete-lined shaft leading vertically down from the intake, and short steel-lined tunnels leading to the Glenmoriston power station. It was the classical arrangement of a short-open-head-race power station placed at the

Fig. 35



SITE OF UPPER MORISTON WORKS

head of the system, and a long tail-race tunnel, in that case a low-pressure tunnel approximately 4½ miles long and unlined. There were surge galleries which also served as access to the upstream end of the tail-race.

In conclusion, Mr Roberts mentioned that the design and construction of the high-pressure systems at Ceannacroc and Glenmoriston were on the same lines as that of the Fasnakyle station described in detail elsewhere in the Proceedings.[86]

Mr R. J. Newport observed that the Author had given as one of the main reasons for the trend towards underground power stations being accentuated in recent years the recent improvements in tunnelling techniques which had resulted in lower unit costs. Those improvements had undoubtedly been considerable, but whilst they made tail-race tunnels cheaper and quicker to construct, he did not see how they greatly affected the cost of pressure shafts which, in some of the examples which had been shown, were greater in length than the tail-race tunnels.

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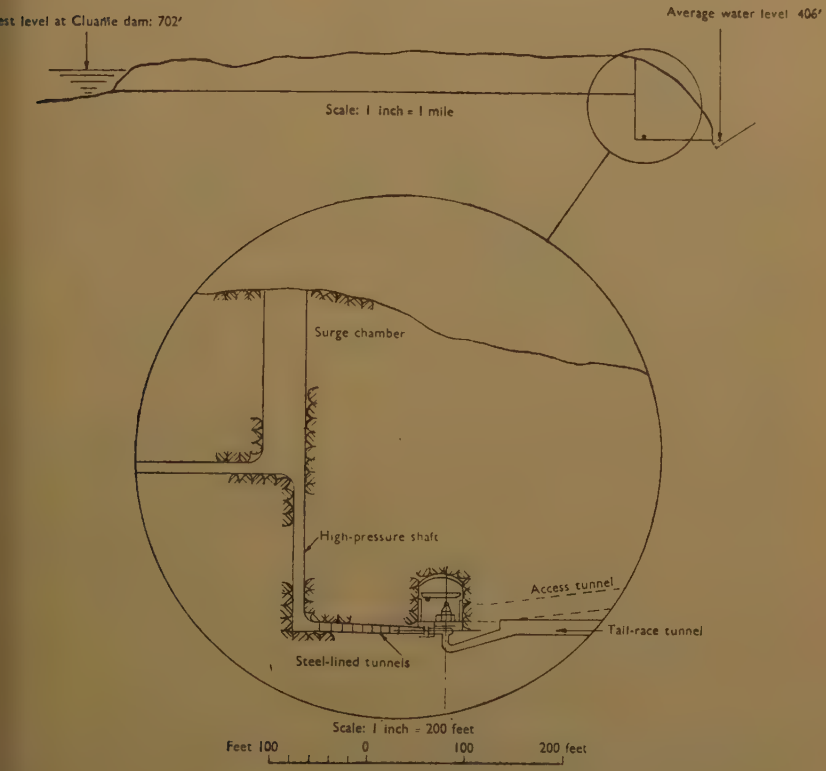
[86] C. M. Roberts, "Special Features of the Affric Hydro-Electric Scheme (Scotland)." Proc. Instn Civ. Engrs, Pt I, vol. 2, p. 520 (Sept. 1953).

Fig. 36

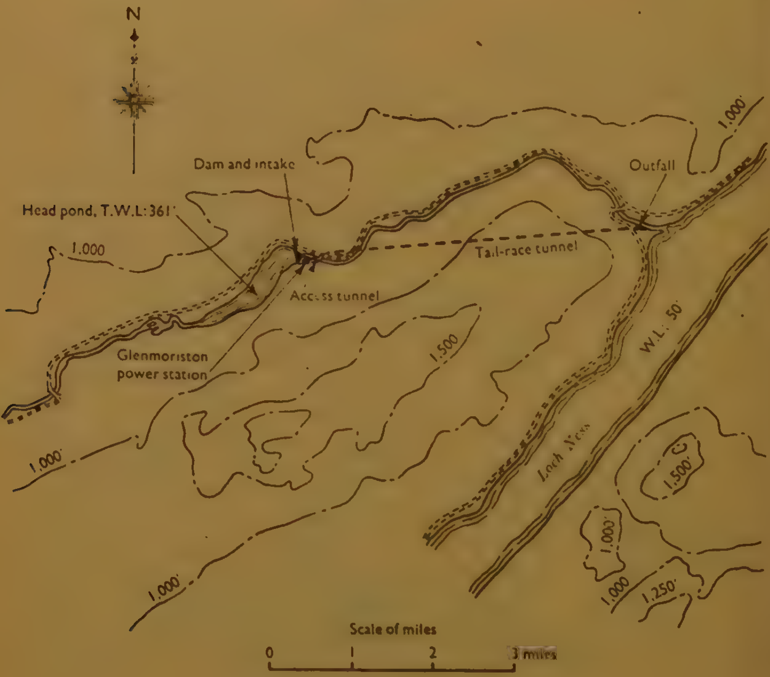


Section of Upper Moriston works

Of the post-war projects listed in Table 5 in the Paper, approximately one-third had vertical shafts and two-thirds sloping. He would be very interested to hear what new techniques had been used to cheapen vertical-shaft construction. The only developments of a major nature of which he had heard had been in South Africa in the very deep mine shafts. The methods which had been developed there in recent years, using mechanical mucking and lining as excavating proceeded, involved considerable expenditure. They seemed to be worthwhile in shafts from 4,000 to 6,000 feet deep, but would hardly be feasible in shafts whose depths were much less. He noticed that one example was given in the Paper, but that particular example would appear to have rather limited applications.

With regard to sloping shafts, his experience had been that they had all the disadvantages of both shafts and tunnels and none of their advantages. Furthermore, sloping pressure shafts would be of circular cross-section, and that would give a very small width of invert to work from. In order to make the best progress and so to reduce the cost, the biggest loading machines possible had to be used, which required plenty of room to change cars behind them. That was not possible in circular sloping shafts; there would be bottlenecks and much hand labour would be needed. The alternative of a flat invert and extra concrete to fill in would also be expensive.

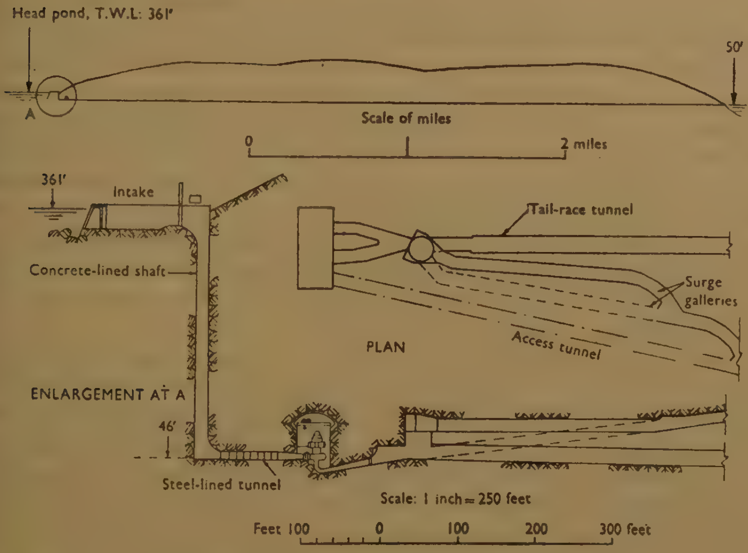
Fig. 37



SITE OF LOWER MORISTON WORKS

Since many of the savings on the tail-race tunnels in the power station would not apply to some of the schemes described, which had only short tail-race tunnels, he felt sure that lower excavation cost could be only one

Fig. 38



PLAN AND SECTIONS OF LOWER MORISTON WORKS

of the reasons for the trend towards underground power stations. Would the Author enlarge on that ?

He had been very much interested to see the Author's flat statement that neither rock nor concrete were completely impervious. That had struck him as being a very sensible admission. He could see no point in impervious concrete being assumed or specified when the chances of getting it were very small indeed. He felt it was much better to face the fact beforehand that concrete or rock would not be impervious and to make arrangements for diverting the water away rather than trying to cure the troubles later on.

Mr I. K. Nixon said that the Author had drawn attention to the rising importance of the physical properties of rock in situ in connexion with the future development of underground hydro-electric power stations. That rising importance applied also to large bridges and concrete dams.

One of the rare applications of the seismic method of determining Young's modulus of elasticity in a rock had been made in 1952 by the company by which Mr Nixon was employed.

The method[87] consisted of utilizing the very close relation which Dr

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[87] P. D. Brown and Jack Robertshaw, "The In-Situ Measurement of Young's Modulus for Rock by a Dynamic Method." *Géotechnique*, vol. III, p. 283 (Sept. 1953).

Reich had found to exist between Young's modulus and the longitudinal wave velocity for a rock. Mr Nixon showed a diagram illustrating the results of a number of tests which, in the main, had been carried out in the laboratory. The range of rocks covered included sedimentary, metamorphic, and igneous rocks. The field work had consisted simply of determining the longitudinal wave velocity. An electrical disturbance had been used to detonate a small charge, and the resulting seismic waves had been picked up by an electrical seismograph placed at a measured distance from the point of the explosion. The interval of time was represented by the period between the instant of the explosion and the arrival of the first vibration impulse, for the longitudinal waves travelled fastest through an elastic medium. Young's modulus could then be interpreted directly from the diagram.

The modulus of elasticity determined in that manner was not materially influenced by minor fissuring because of the long wavelengths involved. Moreover, grouting of the rock joints and fissures, as discussed by the Author, would minimize any non-elastic yield, so that the results predicted by that method would not be appreciably less than the actual values.

In response to the Author's request, Mr Nixon gave the results which had been obtained by Brown and Robertshaw in their tests at Dukan Gorge in Northern Iraq. Massive bedded jointed dolomite had an average modulus of elasticity of the order of $7{\cdot}5 \times 10^6$ lb. per square inch parallel to the bedding planes and $3{\cdot}5 \times 10^6$ lb. per square inch normal to those planes. The corresponding values for thin-bedded finely crystalline limestone (closely jointed and overlying the dolomite) had been found to be appreciably lower—about $1{\cdot}2 \times 10^6$ and less than $1{\cdot}0 \times 10^6$ lb. per square inch parallel and normal to the bedding respectively.

The Author had described the effect of lateral strain on the strength of rock. An advance which Mr Nixon's company had made in that connexion had been the development of an apparatus for carrying out triaxial compression tests at high pressure. For soils it was satisfactory to develop the frictional properties by using pressures of up to about 60 lb. per square inch. With that apparatus it was now possible to develop 1,000 lb. per square inch. He stressed that their conclusions to date were that that pressure was only just sufficient for testing soft rocks, and it would appear that hard rocks would require at least ten times that pressure.

The test consisted of subjecting the rock specimen in a cell to an all-round constant pressure. A closed oil-charged circuit driven by a vane-type pump was used for maintaining the high pressure. The pressure was transferred to water, which was connected directly to the cell *via* copper tubing. The specimen being tested was contained in a metal cylinder. Having developed the required all-round pressure, the specimen was tested in compression under a constant rate of strain. When frictional material was tested in compression, the strength depended partly upon the confining pressure, so that by carrying out a series of tests on the same material with different cell pressures, it was possible to determine the frictional properties as well as any cohesion that might be present.

The danger of explosion of the cell in which the specimen was placed presented a problem which had to be solved. For pressures up to approximately 600 lb. per square inch it was possible to use a Perspex cylinder so that the behaviour of the specimen could be observed during the test. As a result of trials which had been carried out, it had been found that that limit had to be imposed on the Perspex cylinder, otherwise fatigue caused failure to take place. For pressures higher than 600 lb. per square inch, up to the limit of 1,000 lb. per square inch, it was necessary to use a metal cylinder, but it was no great inconvenience not to see the specimen during the test.

The size of specimens which could be tested with the equipment ranged from ¾ inch diameter by 1½ inch long to 4 inches diameter by 8 inches long.

Special equipment had to be assembled for preparing the test specimens, and particular care was required when very soft or friable rocks were concerned. Diamond-impregnated coring drills had been used in the laboratory for obtaining the cylindrical specimens of such rocks.

A considerable range of different materials had been successfully tested with the apparatus described. So far, the strongest material tested, a sandstone, had a cohesion of 29,000 lb. per square foot and an angle of shearing resistance of 41 degrees.

⁂ **Mr J. H. Thornton** observed that, as was stated in the Paper, the concrete filling between the rock and the steel pipe should be as good as possible but he could not see that that had any direct bearing on the length of the steel pipes since it was generally better to place and joint the pipes in one continuous operation and then to place the concrete around them. If that procedure was adopted, then the overall time taken to complete the pressure tunnel would be less than that required for a combined operation. In many cases the pipes were flood-tested hydraulically after being jointed and that could only be done effectively if access around them was possible. He understood that at Kemano one of the pressure shafts was surrounded with prepacked aggregate and grouted as described in one continuous operation after the pipes had all been jointed. That method of concreting shafts and pipes had already been contemplated in Britain and appeared better suited for use in shafts than in horizontal tunnels, because of the difficulty of making grout-tight stunt ends in the latter.

Did the Author know the conditions imposed when welds in pipes were radiographed ? Had all faults to be cut out and made good or were the negatives examined by an experienced inspector and only the worst ones corrected ? The longitudinal joints of the pipes could be machine made

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⁂ This and the following contribution were submitted in writing upon the closure of the oral discussion.—SEC. I.C.E.

and the pipes rotated so that the welding was always downhand, but the site circumferential joints had to be made by hand and, if the pipes were thick, a fully welded joint could take a long time and was expensive. If the pipe was not to be pressure-tested after jointing, the circumferential joints took very little load and a fully welded butt joint was not justified. A cheaper type of joint was used at Fasnakyle and had been described by Roberts.[88]

At Ceannacroc down-hand welding had been used on all the circumferential joints of the straight pipe since the length of pipe was short and it was mounted on rollers that allowed it to rotate. Flanged joints were also expensive and had the disadvantage that it was difficult to pack concrete around them. In the case of pipes where high stresses were possible if all the water load was taken by the steel, what pressures were used in the hydraulic test? What were the advantages of using the special steel for the Oberhasli free pipe since it appeared, from Table 1, that the stress in it was not unduly high? Perhaps it was intended that that steel should have a better resistance to corrosion since it contained chromium and copper. It would be useful if the Author could add a brief note on the various types of protective coating used on the pipelines because that was of importance when considering the head losses in the pipes. A really sound protection was required since a small flaw in the coating might lead to severe local corrosion of the steel which could both give rise to high secondary stresses and finally perforate the pipe. Coatings of zinc and aluminium followed by a zinc-chromate priming paint and three coats of bituminous paint were applied to the pipes at Fasnakyle and it was believed that that would provide a long-lasting protection with the additional safeguard that the metallic coating would protect the steel if the paint was perforated.

Large variations in the value of E for particular rock were given in the Paper and it would be interesting if the methods used to determine the values could be given as well. Determinations from small samples, as described during the discussion, were likely to give higher values than those which would be found by large-scale experiments, as in the Amsteg tunnel. It would also appear that the correct value of E to use in the design of the pipes was that obtained after the rock had been grouted and that could only be obtained either by driving a short experimental heading or by basing the value of E on previous tests in similar rocks. Modern tunnelling techniques demanded good fragmentation of the rock and the effect of the explosives probably caused fissures in the surrounding rock which would give rise to the hysteresis effect illustrated in *Fig. 7*. The stresses in the rock measured by the techniques shown in *Figs 9* and *10* were open to the same criticism and it might be that different values would be obtained if the measurement were to be repeated at different intervals after the tunnel had been excavated and creep had taken place.

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[88] See reference 86, p. 186.

If the stress in the steel lining was calculated for various values of E it would be found that even with small values of E the stress in the steel was quite low and Mr Thornton understood that that had been proved by measurement in poor quality rock. The Author had remarked on the circumferential tensile stress developed in the rock; calculations would show that the tensile stresses in the concrete were even higher and that emphasized the necessity to grout thoroughly the rock surrounding the steel pipes to establish some degree of prestressing.

The grout should be as penetrating as possible and colloidial grout appeared to offer advantages in that respect.

A comprehensive treatment of the design and construction of the Fasnakyle high-pressure tunnel was given in the Paper by Roberts.[89] In that case steel pipes had been used only at the downstream end of the tunnel where the rock cover was not sufficient to balance the water pressure, and the quality of the rock decreased. Those pipes, and those at Quoich and Invergarry, were designed to carry the hydrostatic water load and to withstand the external water pressure. In general, it would be found that the latter condition would govern the pipe thickness, particularly if the pipe shell was not stiffened or otherwise supported. At Ceannacroc and Glen Moriston also the pipes were designed on the same basis, although there was in fact adequate rock cover to take the full water load, but those pipes were required to provide a long water path from the concrete-lined high-pressure shaft to the station and to be self-supporting during manufacture, transportation, and erection.

In introducing the Paper the Author had mentioned that the decision to line or not line a tail-race tunnel with concrete should be left until all the rock could be seen. Where the tunnel was long and friction had an important effect on the output of the station the decision should be made before driving commenced for if that was not the case then either the tunnel had to be driven at the start to a size large enough to remain unlined and a lining placed afterwards if the rock was not suitable, or a smaller tunnel driven first and later enlarged if the rock was suitable. Whichever was done it appeared that the cost and time taken would be greater than if an initial decision was made after considering all the geological information available.

The cross-sections of stations given in the Paper illustrated how the roof of the station had to be designed to suit the rock conditions. A mass-concrete roof in contact with the rock was difficult to analyse but at the same time was extremely strong since before it failed it had to deflect and the deflexion was resisted by the rock mass above it. The Norwegian method of design was more determinate, after the loading conditions had been assumed, and in one case the roof had not been designed until after movements of the rock in the roof had been measured for some time. A false ceiling was usually built below the main roof to hide any damp that might penetrate the concrete and to assist the ventilation of the station.

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[89] See reference 86, p. 186.

Care should be taken in the layout of the station, especially where there were many chambers and tunnels, in order to avoid high stress concentrations in the rock. Where rock pressures were high prior to excavation the additional stresses imposed by much excavation might lead to rock " popping," as had been experienced in one Norwegian station. The " popping " was so severe that work had to cease in some of the chambers until the rock had relieved itself. At Ceannacroc rock " popping " had not been experienced but vibrations from the excavations had caused some rock falls from the walls.

The Author mentioned the cupola alternator-support used at Isère-Arc to enable the turbine to be erected after most of the civil engineering work had been completed. Space in underground stations was limited and Mr Thornton considered that that principle of design should be used more frequently. Modification to the conventional above-ground machine design could lead to substantial savings in excavation and cost.

Mr A. M. Muir Wood observed that there were several interesting ways in which Terzaghi and Richart's theory[90] might be developed. Using the Author's nomenclature, the rock was assumed to be loaded initially with a " hydrostatic " pressure p^*. It was evident from elastic theory that if the periphery of the tunnel indicated in *Fig. 12* (*c*) was to be subjected to a uniform pressure p, then no tensile stresses in the rock would result unless $p > 2p^*$. If, however, it was considered desirable, moreover, to maintain the rock in a " hydrostatic " condition outside a cylinder of radius R, thick-cylinder theory required that $p = \frac{2R^2}{b^2 + R^2} \cdot p^*$, where $2b$ denoted the external diameter of the lining. As $R \rightarrow \infty$, $p \rightarrow 2p^*$ as before. The radial strain at $r = b$ was $\frac{b\sigma t}{E} = \frac{b}{E} \cdot \frac{2R^2}{b^2 + R^2} p^*$ and the stress in a steel lining to the tunnel would in consequence be $\frac{Es}{E} \cdot \frac{2R^2}{b^2 + R^2} \cdot p^*$ where E_s denoted Young's modulus for steel; thus, the total permissible internal pressure might be determined by summation. Suitable choice of the value for E should cater for the possibility that fissures in the rock might be closed as the pressure was applied to the tunnel lining.

If $p_y = Np_x^*$ in *Fig. 12* (*b*), it could be shown that, if the vertical (major) and horizontal (minor) axes of an elliptical tunnel be C and a respectively, then tensile stresses around the excavation would be averted so long as $C > \frac{a}{2} \cdot \frac{(1 - N)}{N}$.

Had the Author any experience of the use of expanding cement grout for prestressing linings? In a sound rock it would appear that the use of such a grout would allow a reduction in the factor of safety in designing the lining.

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[90] See reference 19, p. 171.

Would the Author clarify some points in connexion with *Fig. 7*? An anachronism could be avoided by interchanging "14 Aug. (22h. 35m.)" and "20 Aug. (9h. 25m.)." Was the rock initially stressed to 20 kilograms per square cm. or was there any other explanation of the three parallel inclined lines in the diagram; the shape of the first hysteresis curve would suggest a higher value.

The procedure explained in the text alongside *Fig. 10* for the measurement of initial rock stress by the use of a flat jack was surely valid only if the rock was perfectly elastic. *Fig. 7* indicated how, otherwise, considerable errors would arise on account of hysteresis. Would the Author comment on that?

The Author, in reply, said that the speakers had abundantly fulfilled his desire, when writing the Paper, for more information. In the meantime he had received two interesting illustrations of the Malgovert tunnel of the Tignes Development.[91] *Fig. 39* showed the importance of the reinforcing frames used in crushing rock. *Fig. 40* (facing p. 200) referred to an adit to the main tunnel. There the concrete side-wall had been displaced horizontally by as much as 5 feet and the tunnel invert lifted by several feet.

Fig. 40 was reproduced from a slide shown by the Author when introducing the Paper. Mr Meigh had commented upon it, saying that it gave evidence for a horizontal stress greater than the vertical stress. In fact the whole mountain side through which the adit had been cut was in a state of slow plastic displacement. The Malgovert tunnel afforded a remarkable example of pressure developing and building up steadily in rock. In order to emphasize the opposite case, the Author when introducing his Paper showed a series of slides from the Maggia power stations where the pressure tunnels as well as the long free-surface tunnel (*Fig. 41*) would be left unlined. The designer of the Maggia (1952–56) power stations was Dr A. Kaech, who, some years before, had been responsible for the Handeck steel-lined pressure shaft and later for the Innertkirchen underground power station (1939). Both Innertkirchen and the Maggia tunnels were predominantly in gneiss.

The 6-mile-long pressure tunnel of Innertkirchen was a lined and grouted gallery. The evolution of the technique was quite remarkable.

The discussion had shown the same diversity of cases as outlined in the Paper: the Clachan pressure shaft, described by Mr Banks was steel lined; Mr Guthrie Brown had detailed the grouting progress for a concrete-lined 1,500-foot-long pressure tunnel (600 feet head) and had confirmed the decision to construct that length without steel lining. Mr Roberts, describing the two underground stations on the River Moriston in the North of Scotland, had mentioned that it had been possible to arrange for the 4½ miles of tunnel of the Glen Moriston scheme to be unlined except where weak rock required lining.

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[91] Jean Pelletier, "The Construction of Tignes Dam and Malgovert Tunnel." Proc. Instn Civ. Engrs, Pt III, vol. 2, p. 480 (Dec. 1953).

The information given by Messrs Banks, Brown, and Roberts was extremely valuable and on the whole, as the speakers themselves had underlined, confirmed the figures and facts given in the Paper, with a definite hint that modern designers were ready to accept bolder designs and to place a greater reliance on the ability of sound rock to absorb strains and stresses. That was in line with the conclusions of the Author, whose faith in the ability of rock to absorb stresses and strains was shared by knowledgeable engineers in Great Britain.

Mr Muir Wood's comments on the Terzaghi–Richart theory and the thick cylinder theory confirmed the Author's general statement that the strains and stresses as estimated by both methods could safely be added. (See *Fig. 3*, p. 123, and *Fig. 12* (*b*) and (*c*), p. 137.)

The information volunteered by Mr Ward on tunnels built in London Clay led to the same conclusion. That was a rock formation unlikely to be found often in the design of large water-power tunnels. It was the more interesting to find that that soft material behaved very similarly to hard rock formations. The design put forward by the engineers of the Metropolitan Water Board was based on the same line of thought as that developed by the Author. Mr Ward had mentioned that the average compression strength of the London Clay was about 300 lb. per square inch, which was quite high compared with the full overburden stress of about 80 lb. per square inch. Tests had shown that for short-term loading the lining and the clay behaved approximately as an elastic system with the full water pressure acting on the clay boundary. That was an important statement and every engineer concerned with such problems would look forward with great interest to the full report promised by the Metropolitan Water Board.

Mr Ward had mentioned that when prestressing the tunnel lining the effect of the friction between concrete and rock was extremely high. That remark was very important and should be borne in mind when the methods for prestressing tunnels were further developed. The fact that there was a great amount of friction when any type of tunnel was prestressed had already been noted and mentioned by Mary[83] (1936) when he prestressed (with circular cables) the concrete lining of the pressure tunnel at Marèges (France). He had noted the fact and had admitted that the friction considerably reduced the efficiency of the prestressing.

As an alternative Mr Ward had suggested using jacks together with some plastic medium between the concrete and the rock which reduced the friction to a negligible amount, a method which was being investigated by the Metropolitan Water Board. The grouting method proposed by Kieser would go some way to relieve the effect of friction and it seemed that Mr Ward was well aware of the possibilities of that method also.

It had been noticeable in the discussion that there were two trends of thought concerning anchorages on the steel linings. There was one school of thought, which had been strongly represented, which said that it was not a good thing to anchor the steel lining to the concrete and that it was better to rely on good concreting and on high-pressure grouting. The Author believed that that was probably the right course. There was, however, another school of thought which said that it was necessary to anchor the lining. The reason some people held that point of view was that some linings had buckled, either partially or entirely, under the pressure of water in the rock. The method of using the hedgehog spine had not yet been followed in tunnels ; it had been used only for the pipeline of the dam of Castelo-do-Bode in Portugal, which was anchored in the concrete with that type of anchor.

The Author thought that the question could not yet be finally solved. The drawbacks of the anchorages for compact concreting were obvious but the cases of buckling of steel linings were facts which could not be dismissed. Either the steel lining was anchored and special concreting methods (Prepakt concrete) were used which guaranteed the concrete lining to be homogeneous and compact or the stability of the steel lining against buckling was ensured by successive grouting carried out with increasing grout pressures, as proposed by Mr Guthrie Brown and others. Neither proposal had yet been fully investigated and scope for further research was still open.

Mr Meigh had enlarged upon the recent development of the method known as roof bolting. Roof bolting had been used extensively during the construction of the Forçacava underground power station (Brazil) and the construction of the Kemano Tunnel (British Columbia). The principle on which roof bolting was based was not merely to anchor crushed rock in sound rock, but to introduce a radial component of pressures along the tunnel wall, which tended to re-establish stable rock condition.

Mr Nixon had remarked upon the relation which existed between Young's modulus and the longitudinal wave velocity for a rock. That relation had been shown to exist as early as 1933 and 1934 in several Papers by the Author.[40, 49] The figures given for Young's modulus obtained by the seismic method were generally in line with figures obtained by other methods. At Montperat, Electricité de France used the seismic method for detecting weak rock strata, reflecting the shock waves. Further developments of that method might be helpful for detection of rock strata of different cohesion. It was not known when "small" fissures did not influence the waves and the Young's modulus so obtained and when "large" fissures caused a reflexion of the waves. Problems like those wanted further investigation.

Mr Thornton had pointed out two other problems concerning Young's modulus. All practical data on the modulus available to the Author had been summarized in a former Paper,[31] where data on the moduli and the methods for measuring rock deformation in tunnels were described at length.

The coating used at Fasnakyle, described by Mr Thornton, was the same as the coating used at Innertkirchen and presumably at other stations of the same group (Oberhasliwerke).

Mr Newport had raised a series of questions relating tunnelling methods, tunnelling costs, and underground-power-station design. In a very recent Paper [92] it had been stated that: " Modern highly mechanized blasting techniques have made rockblasting comparatively inexpensive. Among factors which have reduced the cost of rock excavation, attention is called to the use of hard metal rock drills, improved drilling machines and short-delay blasting."

" The cost of construction work inside the rockrooms, on the other hand, has increased, as shown in the accompanying table."

COST OF ROCKROOMS IN SWEDEN

Total volume of installation : cubic metres	Cost of blasting : crowns per cu. m		Inside construction (reinforced concrete) : crowns per cu. m	
	1946–49	1950–51	1946–49	1950–51
10,000	47	40	63	71
20,000	40	33	58	66
40,000	34	28	51	59

The Author thought he was right in connecting tunnelling costs with tunnelling methods.[93] Progress in blasting and in tunnelling methods would keep down excavation cost for vertical and for inclined shafts. For the Cavergno surge shaft (Maggia power system) a vertical borehole had first been drilled downward until it arrived at the level of the lower adit to the main pressure tunnel. Then a platform had been fixed to a cable through the hole and the blasting of a 2-metre-diameter shaft had started in the upward direction. Finally, the 11-metre-diameter shaft (83 metres high) had been excavated by conventional methods, in the downward direction, using the central 2-metre-diameter shaft and the lower adit for spoil removal. Such a procedure made the best of modern methods for rock blasting and spoil removal.

The length of the vertical shaft or the length of an inclined shaft was relatively small in most cases compared with the length of the horizontal tunnels. Underground stations with very short horizontal tunnel sections (Sao Francisco, Picote) were the exception. The cost of excavating the machine hall was also an important item. (See *Fig. 42* showing the excavation for the Peccia power house, Maggia.)

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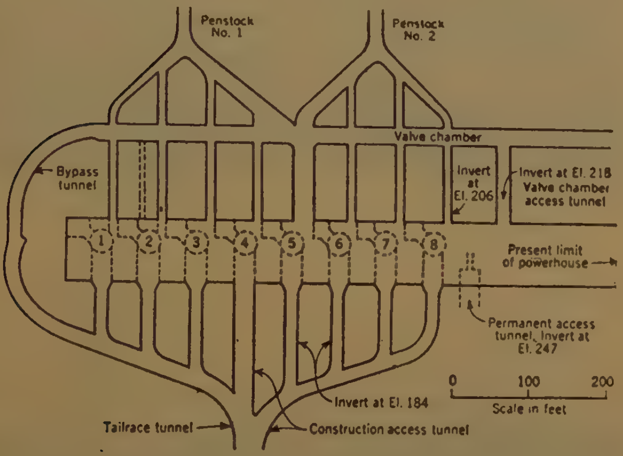
[92] Sten. G. A. Bergman, " Underground Room Construction—Part II." *Engng News Rec.*, vol. 153, p. 37 (9 Dec., 1954).

[93] McNeely DuBose *et al.*, " The Nechako-Kemano-Kitimat Development." Series of 11 articles. *Engineering Journal* (J. Engng Inst. Can.), vol. 37, p. 1381 (Nov. 1954).

It was, therefore, correct to claim that modern developments in the tunnelling methods represented one of the reasons for the development of underground power stations in recent years.

Another tendency in the design of underground stations was to concentrate the maximum power in one station, for a one-step underground development might be cheaper than a more conventional aboveground alternative with one or two steps.

Fig. 43 (*a*)


Penstock No. 1
Penstock No. 2
Valve chamber
Bypass tunnel
Invert at El. 206
Invert at El. 218 Valve chamber access tunnel
1
2
3
4
5
6
7
8
Present limit of powerhouse
Permanent access tunnel, Invert at El. 247
0
100
200
Scale in feet
Invert at El. 184
Tailrace tunnel
Construction access tunnel

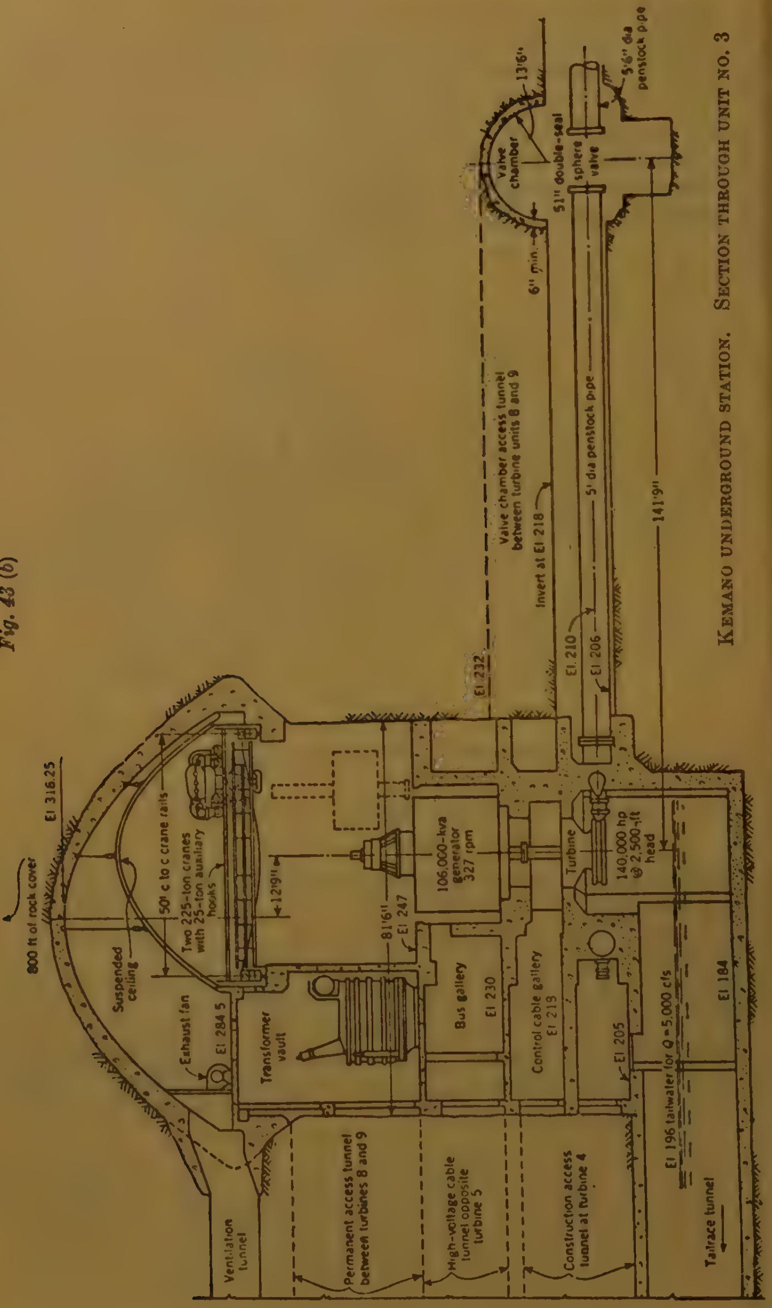
KEMANO UNDERGROUND STATION

The fact that some of the questions asked by speakers could not be answered was proof that considerably more research work still lay ahead.

It seemed fitting to end the discussion with a reference to the powerful Kemano underground station—a very convincing achievement of Canadian engineers—and the Author presented *Figs 43* (*a*) and (*b*) (see p. 200) for comparison with *Figs 27* (*a*) to (*f*).

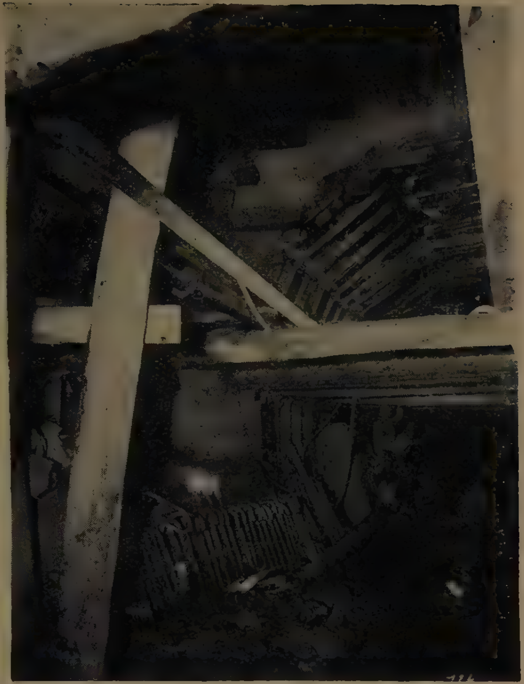
The closing date for Correspondence on the foregoing Paper was the 15th January, 1955 and no contributions other than those already received at the Institution will now be accepted.—Sec. I.C.E.

Fig. 43 (b)


800 ft of rock cover
El 316.25
Suspended ceiling
50' c to c crane rails
Two 225-ton cranes with 25-ton auxiliary hooks
12'9"
Exhaust fan
El 284.5
Transformer vault
81'6"
El 247
106,000-kva generator 327 rpm
Bus gallery
El 230
Control cable gallery
El 219
Turbine
140,000 hp @ 2,500-ft head
El 205
El 196 tailwater for Q = 5,000 cfs
El 184
Ventilation tunnel
Permanent access tunnel between turbines 8 and 9
High-voltage cable tunnel opposite turbine 5
Construction access tunnel at turbine 4
Tailrace tunnel
El 232
Valve chamber access tunnel between turbine units 8 and 9
Invert at El 218
El 210
El 206
5' dia penstock pipe
141'9"
6" min.
Valve chamber
13'6"
51" double-seal sphere valve
5'6" dia penstock pipe

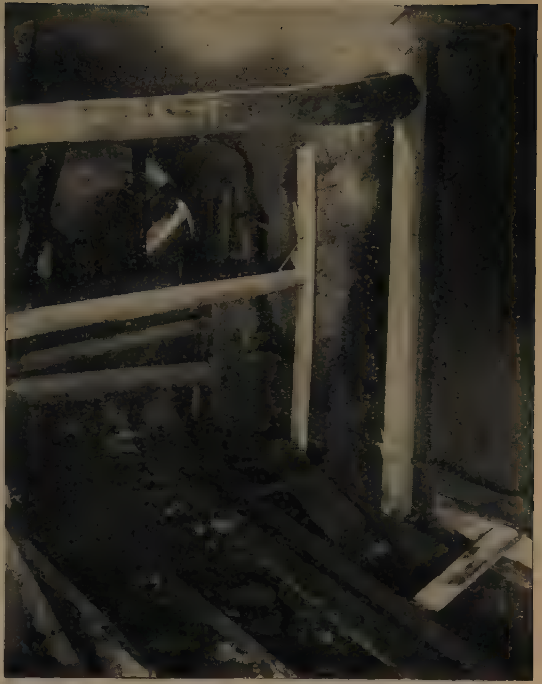
Kemano Underground Station. Section through Unit No. 3

Fig. 39



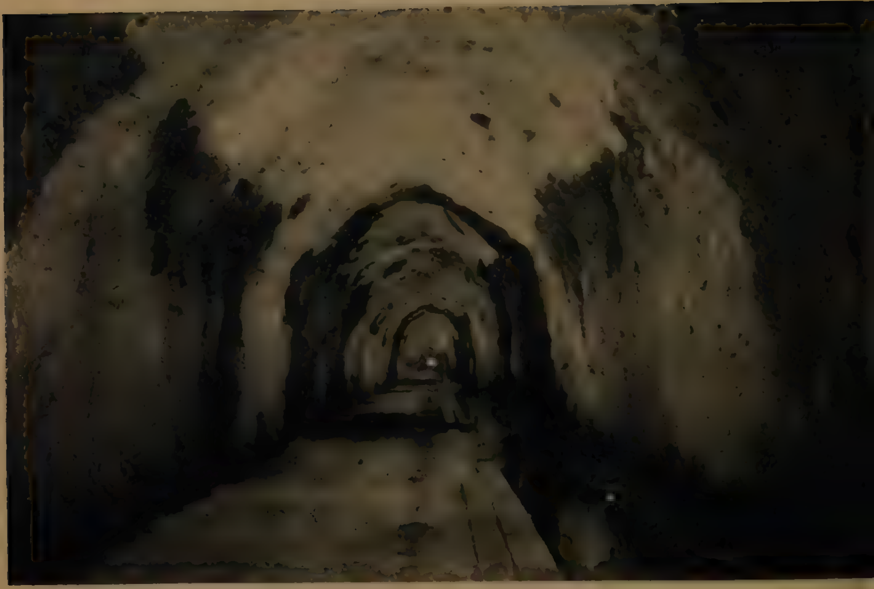
Malgovert tunnel : reinforcing frames

Fig. 40



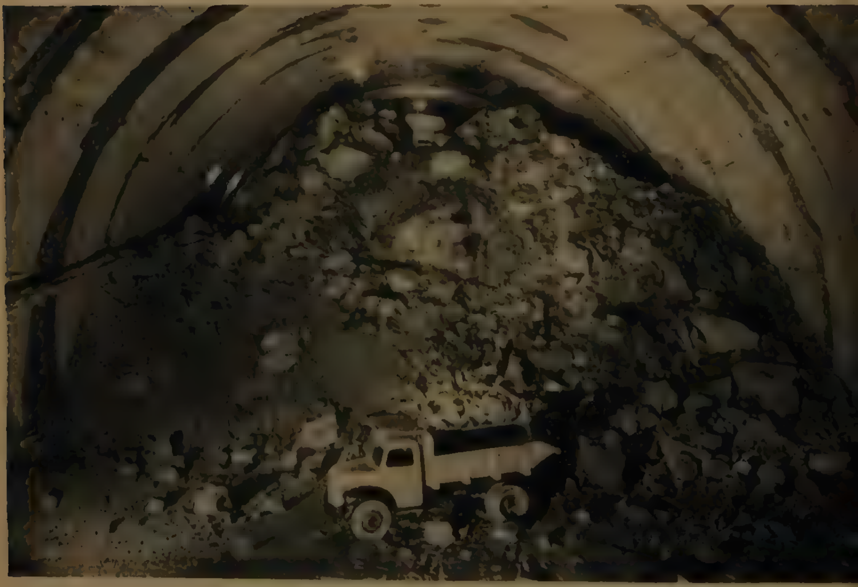
Adit to Malgovert tunnel : horizontal displacement of side wall

Fig. 41



Free-surface tunnel. Maggia scheme

Fig. 42



Excavation for Peccia power house

Paper No. 6012

"The Determination of Tensile Stress/Strain Curves for Concrete"

by

***Joseph Derwent Todd, M.A., D.Phil.**

(*Ordered by the Council to be published with written discussion*)

SYNOPSIS

The Paper compares stress/strain curves obtained from a concrete specimen in pure tension with those obtained from a plain concrete beam subjected to a transverse load.

Previous work has indicated that in the majority of "pure" tension tests there was in fact considerable eccentricity of loading present. A new form of tension test has been devised in which eccentricity of loading can be neutralized by an external bending moment. Stress/strain curves obtained by this means for various mixes of concrete are shown.

Electrical-resistance strain gauges were used to measure the internal strains in plain concrete beams and a method is given for calculating both tensile and compressive stress/strain curves from these results.

INTRODUCTION

THE subject of concrete in tension has always aroused a certain amount of controversy, the chief disagreements arising over what is termed "the strain capacity"; by this is meant the maximum strain that can occur in a specimen before cracking takes place. The strain can either be induced by pure tension or may result from the action of bending.

Squire[1] states that failure of a plain concrete beam in bending does not occur at the load which can be calculated from a stress/strain relationship obtained from a direct tension test, but may very well occur at double that figure. He drew the conclusion that the stress/strain curve would reach a maximum stress at which concrete in pure tension would fail, but, if the stresses were relieved in some way, the concrete would reach this maximum and then extend further at a stress lower than the maximum value. He illustrated this hypothesis with tests that were carried out at Birmingham University and reported by Rao.[2] The specimens used were 30 inches long and had a cross-section of 5 inches by 5 inches reinforced by a central steel rod, the tensile load being applied

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* The Author is Demonstrator, Department of Engineering Science, Oxford University.

[1] The references are given on p. 210.

to projecting ends of the rod. It was found that the strain capacity of the concrete increased with a higher percentage of reinforcement until the latter reached about 3 per cent; thereafter the increase was small compared with the additional reinforcement. The maximum strain reached before failure of the concrete was of the order of 20×10^{-5}. Rao[3] claims that the strain capacity in pure tension is of the order of 5×10^{-5}, but that in bending the limit is raised to about 7×10^{-5}.

Evans[4] has tested a large number of specimens in both tension and bending and he is definitely not satisfied that concrete exhibits properties comparable with those occurring in mild steel above its yield point. In a number of " pure " tension tests he took readings of strain on opposite sides of specimens and found that, in the majority of cases, there was a wide divergence, indicating that eccentricity of loading was present. Thus the tension was not " pure " but was combined with the effects of bending. Tests were carried out on reinforced tension specimens with varying quantities of reinforcement; the experiments being similar to those at Birmingham. The results, however, were quite different, indicating that the quantity of reinforcement changed the strain capacity by very small amounts. Evans found difficulty in determining when a crack had occurred and therefore he developed a technique for locating cracks with a high-powered microscope; this, however, was a long and tedious process. Tests were performed on a number of concrete beams. Evans decided, as a result of all these tests, that there was hardly any difference in the strain capacity of concrete in tension or bending.

Blakey,[5] in a short article, states that an extensive study of strain distribution in concrete results in the conclusion that the first failure occurs when the tensile strain is of the order of 10×10^{-5}. Grassam and Fisher[6] have used electrical-resistance strain gauges on the outside of flat dumb-bell shaped tensile specimens of 3-inch by 3-inch cross-section. In a typical result the maximum strain reached was about 9×10^{-5}.

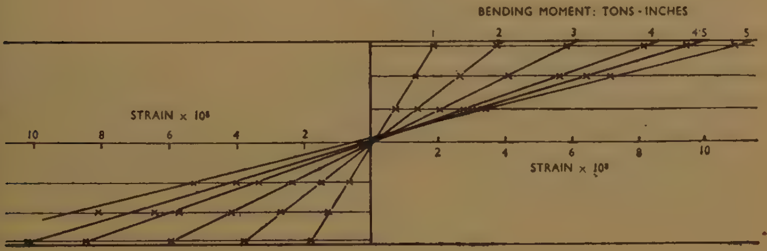
The various results for the " strain capacity," obtained from different sets of experiments, vary quite considerably. Probably a number of these cannot be relied on, since there was eccentricity of loading present during the test. Another cause of error in dealing with a reinforced tensile specimen is failure to locate a crack immediately it has occurred. The Author has made an attempt to compare the stress/strain curves for concrete in tension with those obtained from tests of plain concrete beams.

BEAM TESTS

Throughout these tests electrical-resistance strain gauges were used to measure internal strains in the cross-sections of the beams. It was necessary to have the gauges suitably waterproofed and the technique used was that developed at the Oxford Engineering Laboratory.[7] Five beams of 6-inch-by-4-inch cross-section and 5-foot effective length were tested.

The beams were loaded at middle-third points, so that there was a constant bending moment over the central 20 inches. Strain gauges were placed at suitable levels in order that the strain distribution over the cross-section could be determined. This was found to be sensibly linear in all cases, indicating that plane sections remain plane. A typical diagram of strain distribution over the cross-section is shown in *Fig. 1.* It can be seen from this diagram that, at low loads, the rate of straining on top and bottom surfaces was approximately the same, but, as the bending moment increased, the rate of strain on the bottom exceeded that at the top, indicating a rise in the position of the neutral axis.

Fig. 1



STRAIN DISTRIBUTION OVER BEAM CROSS-SECTION

In four out of the five tests the bottom gauge reached a strain of about 10×10^{-5} and then ceased to record any further increase in strain, although the beam had not quite reached its failing load. This could be attributed to one of two causes. Slip might possibly have occurred between the gauge and the concrete. However, this seems unlikely since the strains reached in each case had approximately the same value. The second explanation is that there was a failure in the concrete at this strain and a minute crack formed somewhere on the lower surface of the beam outside the gauge length. This would probably relieve the strain in the neighbourhood of the crack and account for a stationary gauge reading. In the fifth test, the strain in the bottom gauge increased up to failure of the beam at a higher rate than was compatible with the linear strain over the section. A crack formed across the gauge length in such a way that, when failure occurred, the gauge fractured into two pieces. It was inferred from the above that the maximum strain capacity of a plain concrete beam was of the order of 10×10^{-5}.

CALCULATION OF STRESS/STRAIN CURVES FROM BEAM TESTS

One method that has been used in the past is to assume various stress distributions and, with a knowledge of the bending moment of the beam at failure, to see which of these gives a maximum fibre stress nearest to that obtained from a pure tension test. This method is unsatisfactory because it assumes that failure takes place in tension and bending at the same stress. The Author felt that it should be possible to obtain a stress/strain curve for concrete, both in tension and in compression, from the results obtained from the plain beam tests.

The following symbols are used :—

b denotes breadth of beam.
d ,, depth of beam.
n ,, depth of neutral axis from top surface.
p_c ,, maximum fibre stress in compression.
p_t ,, maximum fibre stress in tension.
e_c ,, strain corresponding to p_c.
e_t ,, strain corresponding to p_t.
M ,, bending moment.

It is necessary to make two assumptions :—(1) Plane sections remain plane; this has been verified by the linear strain distribution over the beam cross-section, and (2) there is no creep during the test.

If the second assumption is true the relation between stress and strain can be represented as : $p = f(e)$.

$$\text{Compressive force} = \frac{\int_o^{e_c} f(e)\delta e}{e_c} \,.\, bn$$

$$\text{Tensile force} = \frac{\int_{e_t}^{o} f(e)\delta e}{e_t} \,.\, b(d - n)$$

Now
$$n = \frac{e_c}{e_t + e_c}\, d$$

Substituting and equating tensile and compressive forces, it can be shown that :

$$\int_o^{e_c} f(e)\delta e = \int_{e_t}^{o} f(e)\delta e \quad . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

The moment arm is given by : $\dfrac{d}{e_t + e_c}\left[\dfrac{\int_{e_t}^{e_c} f(e)e\delta e}{\int_o^{e_c} f(e)\delta e}\right]$

Therefore
$$M = \left[\frac{d}{e_t + e_c}\right]\left[\frac{\int_{e_t}^{e_c} f(e)e\delta e}{\int_o^{e_c} f(e)\delta e}\right]\left[\frac{\int_o^{e_c} f(e)\delta e}{e_c} \,.\, bn\right]$$

$$= \frac{bd^2}{(e_t + e_c)^2}\int_e^{e_c} f(e)e\delta e$$

Differentiating with respect to e_c, substituting from equation (1) and making due allowance for the sign of e_t:

$$p_c = \frac{1}{e_t + e_c} \frac{\delta}{\delta e_c}\left\{\frac{M(e_t + e_c)^2}{bd^2}\right\}$$

Similarly, differentiating with respect to e_t:

$$p_t = \frac{1}{e_t + e_c} \frac{\delta}{\delta e_t}\left\{\frac{M(e_t + e_c)^2}{bd^2}\right\}$$

Similar equations to these have been derived by Prentis.[8]

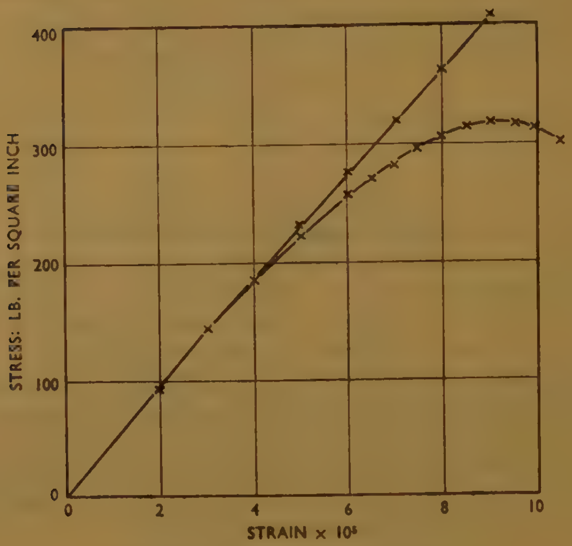
The only requirements for the determination of the compressive and tensile stress/strain curves are the corresponding values of top and bottom strain on the beam at particular values of bending moment. Only one differentiation process is required to determine the required curve from the practical results. This is an improvement on Nadai's method [9] where three differentiations are required to determine the two curves. Differentiating a curve from practical results is not generally very accurate, so that, if the number of processes involved can be kept to a minimum, an improvement should result. It was found that the best method of performing the differentiation process was to plot the curves on graph paper and then to draw tangents to the curves with the aid of a straight piece of glass rod. This method gives an enlarged image of the curve trace and enables the tangent to be drawn with more accuracy than is possible purely by eye.

Compression and tension stress/strain curves were drawn for each of the five beams, a typical one being shown in *Fig. 2*. The tensile curves, for the various mixes used, are shown as the dotted curves in *Figs 3*, *4*, and *5*. In all cases the compressive stress/strain curves approximated to straight lines, with values of E varying from $4{\cdot}4 \times 10^6$ to $4{\cdot}7 \times 10^6$ lb. per square inch. The tensile curves all exhibit the same characteristics. For stresses up to about 200 lb. per square inch the relationship is approximately linear, with a value of E the same as for compression. Above this value the rate of straining increases until a maximum stress is reached at a strain of about 9×10^{-5}. The maximum stresses varied from 266 to 320 lb. per square inch for the various mixed used. Beyond this value there was a slight decrease in stress until failure occurred at a strain varying from $9{\cdot}5$ to $10{\cdot}5 \times 10^{-5}$.

CONCRETE IN TENSION

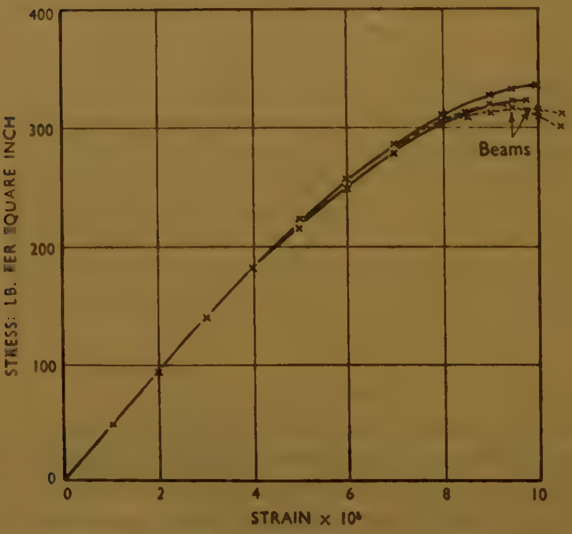
The results from the beam test indicated that for concrete in tension there was a decrease in stress just before failure occurred. If a specimen consisting of pure concrete is tested in tension, failure will occur suddenly when the maximum stress is reached, thus completely masking any effects that might be obtained if the load could be reduced slightly so that a decrease of stress is obtained for an increase of strain. If the

Fig. 2



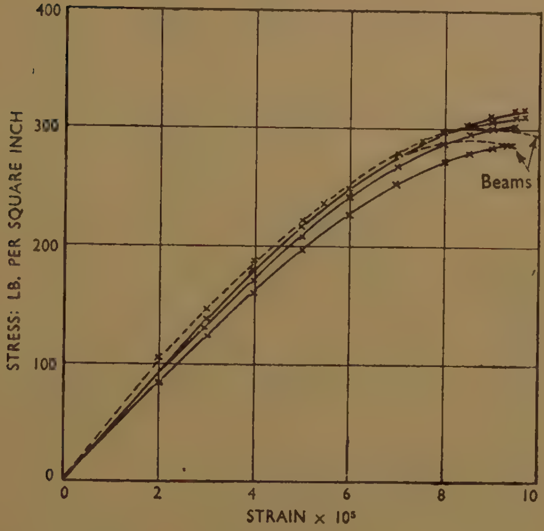
Stress/strain curves from beam test
(water/cement ratio: 0·60)

Fig. 3



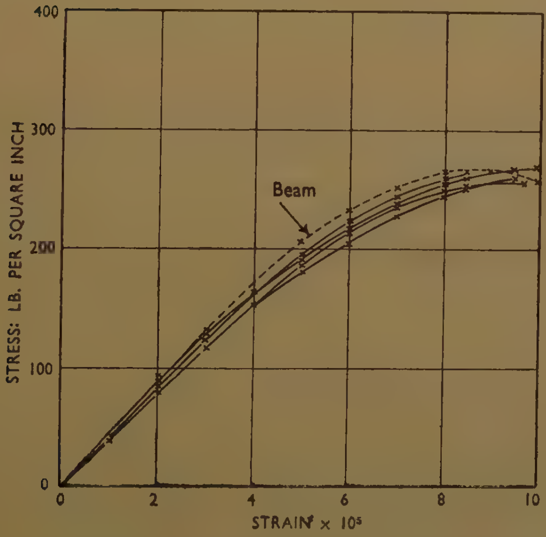
Stress/strain curves for tension and beam tests
(water/cement ratio: 0·60)

Fig. 4



STRESS/STRAIN CURVES FOR TENSION AND BEAM TESTS
(WATER/CEMENT RATIO: 0·65)

Fig. 5



STRESS/STRAIN CURVES FOR TENSION AND BEAM TESTS
(WATER/CEMENT RATIO: 0·70)

stresses in the concrete could be relieved in some way, it might be possible to take observations beyond the maximum stress.

This has been achieved in some measure in experiments already described, by placing a reinforcing bar down the centre of the concrete specimen and applying the load to the ends of the bar. Working on similar lines to this, a new type of tension specimen was devised.

It consisted of a 4-inch-diameter cylinder of concrete 12 inches long, reinforced by a 1-inch-diameter rod 28 inches long, projecting from either end of the cylinder so that the tensile load could be applied to the rod. An electrical-resistance strain gauge with long plastic-covered leads was attached to the middle of the rod and a steel tube $3\frac{3}{4}$ inches long, with an external diameter of $1\frac{1}{2}$ inch and a thickness of $\frac{1}{10}$ inch, was placed over the strain gauge, so that it was concentric with the rod. The annular space between the tube and rod was filled with wax and the ends sealed with hot bitumen. A thin coat of wax was applied to the outside of the tube so that there was no bond between the concrete and the tube. To increase the bond over the remainder of the rod, the surface was knurled for 4 inches either side of the central tube.

The gauge was carefully tested for earth resistance, and then the concrete cylinder was cast round the weigh-bar so that the enlarged portion was in the middle of the cylinder.

It is a simple matter to determine the average stress in the concrete with a specimen of this kind. Readings are taken of load applied by the test machine and strain on the steel bar. The load on the central portion of the steel can be determined from the strain and Young's modulus. This load is subtracted from the total load applied by the machine, and the result gives a measure of the load in the concrete. It should be pointed out that in this method it is essential to have good bond between the rod and the concrete, so that over the central portion the strain is the same in both steel and concrete. The length of the specimen tested was perhaps on the small side, but this was limited by the size of machine available for testing. For measurements to be obtained on the concrete right up to the point of failure it is essential that failure should occur in the central section. The steel sleeve ensured this by reducing the area of the concrete.

Eccentricity of Loading

Eccentricity has probably caused a large amount of error in previous experimenters' work. In all tests three roller-mirror extensometers, reading on a 3-inch gauge-length, measured strain on the central portion of the concrete; the mirrors were set at 120 degrees to each other. In early tests the readings of these gauges differed by as much as 30 per cent just before failure of the specimen. This indicated that it was essential that the eccentricity should be removed by some means.

The reinforcing rod was held in the machine by wedge-action friction jaws. Adjustment of these to remove the eccentricity proved to be an almost impossible task. The only other method available was the application of an external bending moment, and the following technique was found to be most successful.

The specimens were all tested in a vertical position. A loop of sufficient size to pass round the specimen was made in each end of a piece of "Bowden" cable. The two loops were placed in position so that each occupied a middle-third point along the cylinder. The wires were taken away horizontally and passed over pulleys, and weights were hung from the middle of the wire. By this means a constant bending moment could be applied over the middle-third of the specimen by adding weights to the wire. It was found convenient to have three sets of this equipment, each placed so that the load was in the same plane as the external extensometer. If a particular extensometer was reading low it was only necessary to add some weight to the system in line with it. This would increase the reading of the extensometer and at the same time lower the readings of the other two. With practice it became a simple matter to adjust the extensometers to the same strain reading as each increment of load was applied to the specimen. So long as the external extensometers are reading the same, the central portion of the specimen must be under the action of pure tension without any bending.

Once loading had commenced, the plane of bending due to the eccentricity remained fairly constant; in some cases as much as 70 lb. and 40 lb. had been added to two of the loading systems when failure occurred.

TESTING

Ten specimens were tested by this means, varying water/cement ratios being used corresponding with those used for the beams.

The agreement between the roller-mirror extensometers and the electrical-resistance strain gauge was generally to within 2 to 3 per cent; the maximum difference in any test was 5 per cent. This indicated that the bond was sufficient to transmit the load to the concrete and that the strain distribution was constant over the centre section.

When failure was about to take place a particular applied load would be reached at which all the extensometers would register the same reading. A further increase of load would cause a sudden creep in the gauge on the steel, indicating that failure had started. The word "started" is used, since failure always occurred at one side of the specimen before the other. One of the external extensometers would start to creep slowly and then the rate of creep would increase. If observations were taken on the other two extensometers, it was found that the readings were at first either stationary or else had a small negative creep; after a short time a positive creep would take place at increasing speed. It was found impossible to add weights at a sufficient rate to counteract this creep, since the failure was completed in less than 15 seconds. From this it would appear that failure always occurred at one side of the cylinder before the other, then gradually spread through to the other side until the failure was complete. This was probably due to some slight flaw in the concrete at one side, and was completely independent of the plane of the eccentricity.

Complete failure was evident from the readings of the electrical-resistance strain gauge, since this showed when the load was being carried by the steel alone. No crack was visible to the naked eye and it was necessary to increase the load considerably before the position of the crack was located. In all cases the crack occurred in the central portion of the specimen.

Comparison between Pure Tension and Beam Tests

In *Figs 3, 4*, and *5* the tensile curves obtained from the beam tests have been superimposed on the corresponding pure-tension curves. The agreement is good considering the method used for obtaining the beam curves. The difference in maximum stress is small and the shapes of the curves are similar. The pure tension curves do not exhibit the slight falling off in stress near failure that was a feature of the beam-test curves. This is probably due to the fact that failure always started at one side of the specimen and, once this had occurred, it was not possible to obtain any further useful readings. The maximum strain reached in the beam test is about 5 per cent greater than in the tensile test.

Thus it would appear that the maximum stress and strain that concrete can withstand is approximately the same in pure tension and bending; the maximum strain being of the order of 10×10^{-5}.

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The Paper, which was received on the 1st March, 1954, is accompanied by one photograph and five sheets of diagrams, from which the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution before the 15th July, 1955. Contributions should not exceed about 1,200 words. —SEC.

Paper No. 6025

"Bond Stresses in Prestressed Concrete from X-Ray Photographs"

by

*** Professor Rhydwyn Harding Evans, D.Sc., Ph.D., M.I.C.E.,**

and

Gerald Wilson Robinson, B.Sc., Ph.D., Stud.I.C.E.

(Ordered by the Council to be published with written discussion)

SYNOPSIS

Distributions of strain in steel and concrete, slip, and bond stress are measured by a radiographic technique which does not interfere with the bonding conditions in the transmission lengths of uniformly prestressed concrete columns, and in the vicinity of cracks in pretensioned prestressed concrete beams. The action of creep and shrinkage on the distribution of these quantities is investigated, and a relation is deduced connecting the end slip or "pull-in" of the reinforcement with the transmission length. Very large localized bond stresses are recorded in the vicinity of cracks.

INTRODUCTION

TESTS to determine the bond between concrete and steel had been made as early as 1876 by Thaddeus Hyatt, and the results of these early investigations, summed up by Abrams[1] in 1913, indicated that the internal anchorage which occurs naturally as bond between concrete and steel is often insufficient to develop the potential strength of a flexural member; it therefore requires supplementing with some form of external anchorage. The provision of this anchorage in the form of a hook at the end of a piece of tensile reinforcement is so cheap and simple that most designers have been content to leave the resistance to slip "hanging from the hook," and the effects of bond have mainly been of academic interest.

Freyssinet succeeded in prestressing concrete in 1928, but the anchorages he required for his $\frac{5}{8}$-inch-diameter high-tensile-steel bars were complicated and costly. However, many Continental engineers anticipated that when cold-drawn wires of less than $\frac{1}{4}$-inch diameter were introduced the full prestress might be developed by the bond resistance of a fairly short embedded length of wire, thus dispensing with external anchorages; thus the bond between concrete and steel again became a practical issue.

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* Professor Evans is Professor of Civil Engineering, Leeds University, and Mr Robinson, at the time of writing the Paper, was a Research Fellow in the Department of Civil Design, Harvard University.

[1] The references are given on p. 235.

Measurements of bond stress are made by first determining the strain in the steel. There are two general methods of doing this. Mains [2] has devised a method, which is a development of the technique used by Glanville,[3] which involves attaching strain guages to the *inside* of a specially prepared hollow steel reinforcing bar and running the leads out at the ends. This technique allows the measurement of strains whilst causing no disturbance to the bonding conditions, but it is necessarily restricted to bars of large diameter, and is, therefore, not applicable to the wire reinforcement used in pretensioned work. All other methods depend for their effect on making a hole in the concrete and measuring displacements of the exposed steel by normal laboratory techniques. These methods have the disadvantage of eliminating concrete from the surface of the steel with a consequent destruction of bond; this destruction of bond becomes more serious as the diameter of the wire is reduced. Furthermore, when these holes occur in the tensile zone of a beam, they may upset the whole stress pattern by acting as crack initiators, causing cracks to open prematurely or in locations where they normally would not occur.

The necessity for measuring strains in wires without disturbing the surrounding concrete mothered the invention of a radiographic technique which was used in the investigation described in this Paper. At its simplest, radiography consists of casting shadows. The radiographic strain-measuring technique consists of measuring the displacement of shadows thrown by markers which absorb radiation differentially, and from this arriving at an assessment of the displacement of the original markers. At its present state of development the technique measures strains to an accuracy of 3×10^{-4}, but it is capable of much refinement.

THE RADIOGRAPHIC STRAIN-MEASURING TECHNIQUE

The radiographic strain-measuring technique involves placing small lead markers in slots formed in the reinforcement. Concrete is cast round them in the usual way. The positions of the markers are recorded on an X-ray photograph which is measured by a travelling microscope. The radiographic conditions for obtaining a satisfactory film are critical, depending on the quantity and nature of the radiation, the placing of the specimen, its size and the materials that compose it, the degree of shielding and filtration employed, the type and condition of film, and the processing technique.

Of the several X-ray tubes that were investigated, an S.R.T.2 medical diagnostic tube with a tungsten target in a G.E.C. 200-kilovolt unit, produced the most satisfactory results, requiring an exposure of 2 minutes when working at 135 kilovolts with a filament current of 12·5 milliamperes. This tube had a source area of one square millimetre and, by placing this at a minimum distance of one metre from the specimen to be radiographed, the area of penumbra on the film could be restricted to a satisfactory extent.

Slots for locating the lead markers in the reinforcement were cut by a milling machine, using high-speed cutters 2½ inches in diameter and 0·005-inch thick. A preliminary investigation showed that a slot of depth 0·012 inch reduced the ultimate strength of 0·08-inch-diameter wire by 7 per cent; this was considered a satisfactory compromise between adequate slot depth and wire strength. For larger wires the depth is less critical.

Markers were cut out of sheet lead that had been rolled out to a thickness of 0·005 inch. They were cut by a sharp blade on a piece of plate glass to a size of approximately 2 millimetres by 1·5 millimetre. Thus, when the marker was placed in its slot, approximately 1 millimetre of it projected. The insertion of the marker was accomplished with the aid of a pair of fine forceps; the process demanded a steady hand, considerable skill, and good illumination.

The film used was Ilford industrial B, combining moderate speed with fine grain. It was processed under standard conditions and measured in a metrological microscope accurate to 0·0001 inch. Changes of humidity produce considerable dimensional changes in this film. These were to a large extent eliminated by storing and conditioning the film under conditions of constant humidity, but film shrinkage remains the limitation on the accuracy of the technique. A non-chemical photographic process known as xeroradiography, developed recently by McMaster,[4] promises to remove this limitation by using an insulating plate in place of the film. This should increase the range of the technique by improving its accuracy.

Fig. 1 shows a photomicrograph of a portion of an X-ray film. The broad light band traversing the plate is the shadow cast by a 0·08-inch-diameter wire. This wire has moved to the left through the concrete, shearing off the portion of the lead marker which projected beyond the slot into the concrete. The slip of the steel relative to the concrete, therefore, may be measured directly as the distance between the portions of a marker embedded in the steel and in the concrete, and steel strains may be measured by the changes in distance between portions of adjacent markers embedded in the steel. The ease and accuracy with which slip may be measured makes it a fundamental quantity of this investigation.

TEST SPECIMENS

Table 1 gives the elastic constants of the various wires the bond characteristics of which were investigated.

The figures refer to tests on wires with ten slots 0·005 inch wide milled into the surface. For the 0·08-inch-diameter wire the slots were 0·012 inch deep and for all other wires, the slots were 0·014 inch deep.

TABLE 1

Diameter: inch	Diameter: millimetres	Surface	Modulus of elasticity: Initial tangent: lb. per square inch	Modulus of elasticity: Secant 200,000 lb. per square inch: lb. per square inch
0·08	2	Plain	$29{\cdot}0\times10^{6}$	$29{\cdot}0\times10^{6}$
0·2	5	Plain	$27{\cdot}8\times10^{6}$	$27{\cdot}4\times10^{6}$
0·2	5	Indented	$28{\cdot}2\times10^{6}$	$27{\cdot}8\times10^{6}$
0·275	7	Plain	$29{\cdot}2\times10^{6}$	—

Ferrocrete rapid-hardening cement was used for all the concrete specimens of which details are given in Table 2.

TABLE 2

Specimen	Mix	Cube strength at 7 days: lb. per square in	Modulus of elasticity secant 3,000 lb./sq. in: lb. per square inch	Shrinkage strain: 7 days	Shrinkage strain: 1 month
Columns	1:1:2	8,600	$4{\cdot}72\times10^{6}$	$3{\cdot}7\times10^{-4}$	$5{\cdot}1\times10^{-4}$
Beams	1:1½:3	6,300	$3{\cdot}90\times10^{6}$	$2{\cdot}6\times10^{-4}$	—

The design of the concrete members was conditioned by the radiographic technique. It was necessary to radiograph columns before the prestressing force had been released; this required that the column and the mould, which are illustrated in *Fig. 2*, should be fairly portable to enable them to be placed beneath the X-ray unit. For good definition it was also desirable to limit the thickness of the column and to place the reinforcement as close to the sides as possible; the design finally adopted was for columns of 1½-inch-by-1½-inch-by-3-feet-6-inches, reinforced with either six 0·08-inch or one 0·2-inch-diameter wire. When these wires were stressed to 82 tons per square inch, a prestress of 3,000 lb. per square inch was induced in the concrete. The same limitations restricted the size of beams, and a fairly small section modulus was desired since the reactions arising from loading would have to be withstood by a specially built testing machine. These beams were 6 feet 6 inches long, of 8-inch-by-8-inch T-section with a web 5 inches by 1 inch. They were reinforced by either eighteen 0·08-inch-diameter wires, or by twelve 0·08-inch- and one 0·2-inch-diameter wires. Stressing these wires to 75 tons per square inch induced a uniform prestress of 1,000 lb. per square inch in the concrete; this low value caused cracks to open at an early stage of the loading cycle.

THE NATURE OF BOND

Quantities influencing bond may be divided into two categories, the adhesive and the frictional, although the distinction is largely one of convention. Adhesion is a loose term to describe an initial resistance to slip, that is, a resistance that is present only when slip is very small. Of the theories advanced to explain it, micro-mechanical locking appears to be the most reasonable. This postulates that the resistance to slip is given by the shear strength of the fine particles of concrete that have been forced into micro-indentations in the surface of the reinforcement. When a slip comparable with the size of the micro-indentations takes place, the adhesive resistance disappears.

Frictional Resistance.—This may be assessed by multiplying the radial pressure by the appropriate coefficient. Some coefficients given by Armstrong [5] show the influence of grease and rust on the surface. The thickness of the film as determined by microanalysis provides a convenient standard for the degree of greasiness, and Wormald [6] has prepared photomicrographs to standardize the degree of rusting.

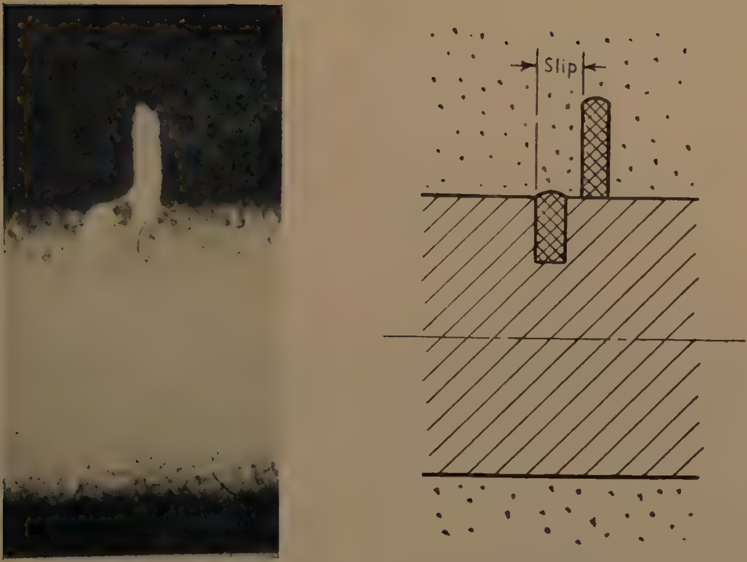
Mechanical Locking.—This is a phenomenon similar to the micro-mechanical locking described previously, except that the indentations are much greater. The indentations on the 0·2-inch-diameter wire were of 0·1-inch diameter and were spaced at the rate of ten per inch. Therefore, assuming a shear strength of 2,000 lb. per square inch, the increase of bond resistance from the locking may be expected to be 160 lb. per square inch.

Dilatancy.—This effect, investigated by Jenkyn, is a resistance to slip arising from the wedging action of the fine particles produced after an initial slip has taken place.

Wedge Action.—This is a frictional resistance resulting from the setting up of radial strains in locations where the longitudinal strain is changing. An analysis by Marshall [7] gives the angle of the wedge as $\tan^{-1}(3{\cdot}24\times10^{-5})$ for the columns with 0·08-inch-diameter wire, and assuming that the concrete surrounding the wires will withstand a hoop tension of 1,000 lb. per square inch, the maximum contribution towards bond resistance is seen to be about 200 lb. per square inch.

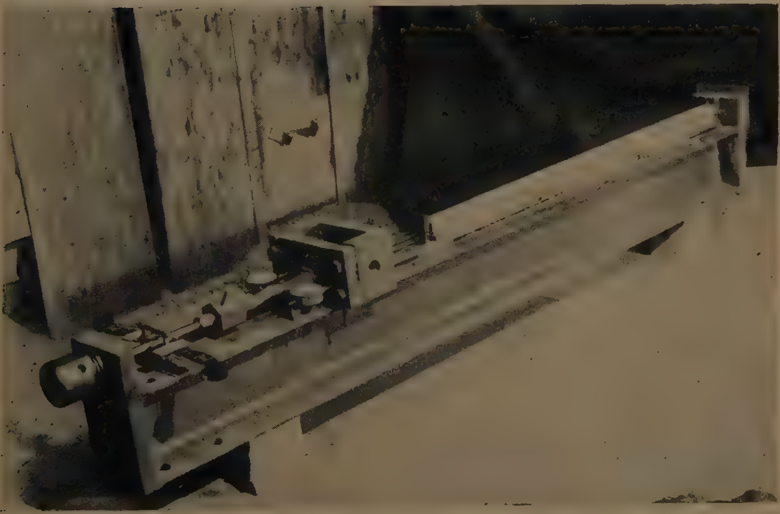
Tangential Friction.—Tangential friction is a localized frictional resisttance which occurs adjacent to cracks in beams. Because the deflexion of a cracked beam results mainly from the opening of cracks, the reinforcement makes a sudden change of direction at each crack, and a radial component of the tension in the reinforcement will, therefore, act upon the concrete adjacent to each crack. This radial force is of value $\frac{Mw}{2a^2}$ where M is the bending moment in the vicinity of the crack of width w and a is the lever arm of the tensile reinforcement. This radial force may be said to act over such an area as to give rise to a stress equal to the

Fig. 1



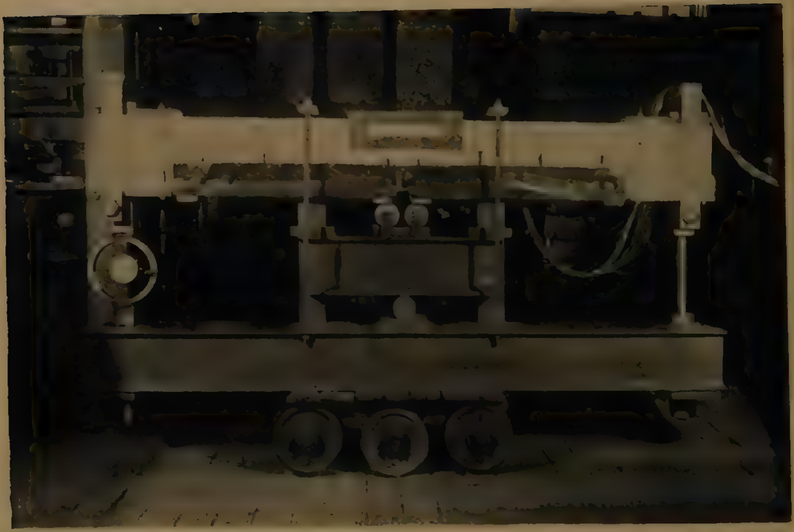
Photomicrograph of portion of X-ray film

Fig. 2



Column and mould for prestressing tests

Fig. 9



LOADING FRAME FOR TESTING BEAMS

crushing strength of the concrete. For the beams under test there was an increase of bond resistance of 250 lb. per square inch over a distance of 0·15 inch from each crack of width 0·01 inch; this distance increased rapidly with increasing crack width.

Grease Film on Wires.—The actual condition of the surfaces is open to subjective interpretation. The surfaces are often defined as polished, slightly pitted, or badly pitted. For this investigation it was considered desirable to standardize the degree of greasiness, and a convenient method of effecting this was by measuring the thickness of the grease deposit or film. The thickness of the film was obtained by measuring the weight of grease occurring on a given surface area. A sample of wire weighing about 5 grams was weighed in an accurate balance reading to 10^{-6} gram. It was then cleaned by successive immersions in ether and carbon tetrachloride for 20 minutes, the solvent being agitated; the loss in weight was equal to the weight of grease removed by the solvent. Wire A was cleaned with a rough texture cloth and solvent; wire B with white cloths only; wires C and D were obtained from industry in the condition in which they are normally used for pretensioned members. Some of the results obtained are shown in Table 3.

TABLE 3

Wire	Average surface area: square centimetres	Average loss in weight: milligram	Loss per unit area: milligram	Thickness of film:		Maximum bond stress: lb. per square inch
				centimetre	inch	
A	6·61	0·015	0·00227	$0{\cdot}002\times10^{-3}$	$0{\cdot}8\times10^{-6}$	1,630
B	6·62	0·190	0·0287	$0{\cdot}024\times10^{-3}$	10×10^{-6}	1,500
C	14·86	2·125	0·1430	$0{\cdot}125\times10^{-3}$	49×10^{-6}	850
D	16·68	0·817	0·0490	$0{\cdot}042\times10^{-3}$	17×10^{-6}	1,290

These figures illustrate the importance in practice of thoroughly cleaning the wires unless other means, such as crimping or surface indentations, are adopted to increase the overall bond strength.

CONDITIONS IN PRESTRESSED COLUMNS AT RELEASE

Concrete Strains

Concrete strains were measured by a method devised by Evans[8] which involves measuring the distance between pins embedded in the concrete with a travelling microscope. The transmission length is defined as the embedded length of wire required to develop the full prestress in the concrete, and this may be determined from a graph showing distribution of concrete strains after release. Table 4 summarizes transmission lengths as estimated by this method.

TABLE 4

Wire diameter:		Initial prestress in steel:	Transmission length on release:	
inch	millimetres	lb. per square inch	inches	diameters
0·08	2	$17{\cdot}5\times10^4$	6·7	84
0·2	5	$16{\cdot}4\times10^4$	24·0	120
0·275	7	$14{\cdot}6\times10^4$	50·0	182

Steel Strains and Slip

Graphs of steel strains and slips, shown in *Fig. 3* were obtained by measuring X-ray films showing the displacements of markers. Curves A, B, C, and D were prepared by exposing the film when varying percentages of the prestressing force had been released. For convenience in plotting, zero strain is taken as the strain associated with the initial stress in the steel. The ordinate to the slip curve at the end of the column is the end-slip or " pull-in " of the reinforcement; this was also checked externally by measuring the displacement of scratches on the reinforcement at the end of the column. Transmission lengths obtained from these graphs are shorter than those observed by other investigators; this is because the radiographic technique permits the measurement of strain at a time more nearly approximating to the theoretical $t = 0$, and time effects cause a rapid increase in the transmission length.

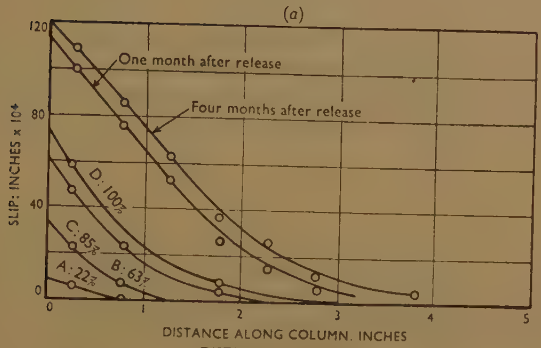
Bond-Stress Distribution

Bond-stress distribution curves were prepared from steel-strain curves by a process of graphical differentiation, and the scales determined from particular values. There is a large personal factor involved, but results obtained by replotting were found to be consistent to 6 per cent, which is reasonable. Since it was feared that the actual measuring slots might have some influence on the bond, a set of columns was cast bearing the minimum number of slots necessary for an evaluation of bond stress; this was less than half the number on a normal column, but it was not possible to detect any altered bonding conditions.

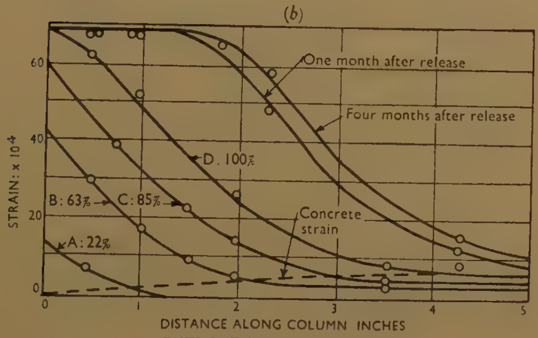
Factors which might increase the bond of a slotted wire are:—

(1) Mechanical locking at the edges of a slot.
(2) A crimping effect arising from the eccentric load in the wire.
(3) Additional scratches and bends that the wire might receive during its additional machining.

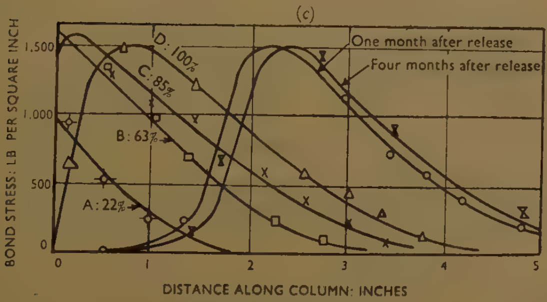
Fig. 3



DISTRIBUTION OF SLIP



DISTRIBUTION OF STEEL STRAINS



DISTRIBUTION OF BOND STRESS

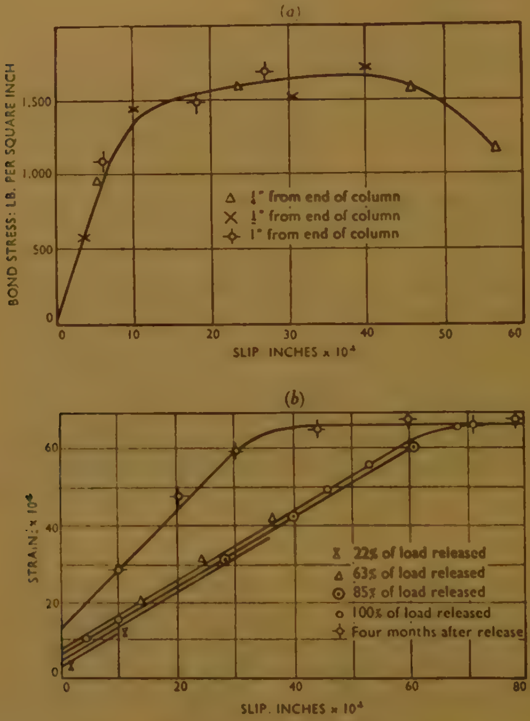
Slip, strains, and stress in column G1
(Six 0·08″-dia wires)

Factors which might reduce the bond are :—

(1) Elimination of the bond by the projecting lead markers.
(2) Lubrication of the sliding surfaces by the lead.

Either all these factors are of negligible proportions, or they balance one another. The fact that observed bond stresses were higher than those recorded by Marshall[7] results from the fact that the concrete round the wires was not disturbed.

Fig. 4



Slip and bond stress, and slip and steel strains in column G1
(Six 0·08"-dia. wires)

Bond stresses occur wherever stresses in the steel and concrete are changing in the opposite sense. This implies that strains also are changing in the opposite sense, which gives rise to relative movement, or slip, between the concrete and the steel. Thus, bond stress is always associated with slip and the commonly quoted definition of adhesion as the bond stress for zero slip is not valid, although the slips are very small. Because slip is s general an accompaniment of bond stress, the influence of the magnitude of the slip on the bond stress is a matter of great interest. Glanville [3] did some early work in this field but he was able to measure slips only at the ends of his members, where conditions are rather specialized. *Fig. 4 (a)* shows the relation for various points in column G1; the curve has a striking similarity to the curve obtained by testing a steel specimen in shear. The curve relating bond stress to slip is found to be unique for any particular member, but more work will have to be done before it is possible to produce a universal relation involving all variables.

Distribution of Bond Effects

Let S denote steel strain at distance x from the free end of the column
δS ,, small decrease over increment δx
S_o ,, strain in steel beyond the transmission length
S_1 ,, strain in steel before release
Δ_1 ,, slip at end of column
Δ ,, slip at distance x
$\delta\Delta$,, small decrease of slip over increment δx
E ,, modulus of elasticity for steel
E_c ,, modulus of elasticity for concrete
A_c ,, area of concrete section per wire of nominal diameter d

The load in the steel at distance x $= SE\frac{\pi d^2}{4}$

The load in the steel when all transfer has taken place $= S_oE\frac{\pi d^2}{4}$

Load transferred at x $= (S - S_o)E\frac{\pi d^2}{4}$

Load transferred at $x + \delta x$ $= (S - S_o - \delta S)E\frac{\pi d^2}{4}$

Maximum load in the concrete $= S_oE_cA_c$

Load in concrete at x $= S_oE_cA_c - (S - S_o)E\frac{\pi d^2}{4}$

Load in concrete at $x + \delta x$ $= S_oE_cA_c - (S - S_o - \delta S)E\frac{\pi d^2}{4}$

Average concrete strain over δx $= (S_o - \left(S - S_o - \frac{\delta S}{2}\right)\frac{E}{E_cA_c}\cdot\frac{\pi d^2}{4}$

Now slip is the relative displacement of steel and concrete, that is, it is the difference between the absolute movement of the steel and the concrete, therefore:

$$\delta\Delta = \left(S - \frac{\delta S}{2}\right)\delta x - S_o\delta x - \left(S - S_o - \frac{\delta S}{2}\right)\frac{E}{E_cA_c} \cdot \frac{\pi d^2}{4}$$

which is, neglecting the second order of small quantities:

$$\delta\Delta = (S - S_o)\left(1 - \frac{E}{E_cA_c} \cdot \frac{\pi d^2}{4}\right)\delta x \qquad = C_3(S - S_o)\delta x$$

$$\text{therefore,} \quad \frac{\delta\Delta}{\delta x} = C_3(S - S_o) \qquad = \frac{d\Delta}{dx} \text{ in the limit.}$$

Now from *Fig. 4* (*b*) it is seen that, during release, there is a straight line relation between slip and strain, which may be expressed:

$$\Delta = C_1(S - S_o)$$

$$\text{Differentiating} \quad \frac{d\Delta}{dx} = C_1\frac{dS}{dx}$$

$$\text{therefore} \quad \frac{dS}{dx} = C_2(S - S_o)$$

$$\text{or} \quad \frac{ds}{(S - S_o)} = C_2dx$$

$$\text{therefore integrating} \quad (S - S_o) = (S_1 - S_o)e^{-C_2x}$$

which is an exponential relation expressing the distribution of steel strains in the transmission length. A relation of similar form has been deduced on a stress basis by Mautner, and on a bond stress basis by Wormald. However, a slip basis has the advantage of using quantities that are directly measurable.

$$\text{Now} \quad \Delta = C_1(S - S_o)$$

therefore the distribution of slip may also be expressed as an exponential relation of the form:

$$\Delta = \Delta_1e^{-C_2x}$$

where Δ_1 is the slip that occurs at x=0, and is equal to the end slip or "pull-in" of the reinforcement.

Bond stress distributions are obtained by graphically differentiating steel strain curves; therefore, bond stress distribution may be expressed:

$$f_b = f_1e^{-C_2x}$$

When $x = 0$, f_b also $= 0$, but in a very short distance along the column the maximum bond stress f_1 will be attained if the release of the prestressing force has been rapid, so this value is inserted at the lower limit.

At the end of the transmission length, slip and bond strength will both equal zero, and the steel strain will have attained its limiting value. That is, when $x = l$, $\Delta = 0$, $f_b = 0$ and $S = S_o$. The equations for slip, strains, and bond stress show that these quantities become smaller proceeding along the column, but they never attain their limiting values, so it is necessary to postulate some limiting value for these quantities beyond which the exponential relation does not hold. The actual value selected will be arbitrary, and designed to give good agreement with the measured transmission lengths.

TABLE 5

Column	Diameter of wire :	Transmission lengths in inches and diameters							
		Measured :		Calculated					
				ratio $\frac{\Delta_0}{\Delta_1}=1$ per cent		ratio $\frac{\Delta_0}{\Delta_1}=2$ per cent		ratio $\frac{\Delta_0}{\Delta_1}=3$ per cent	
	inch	inches	dia.	inches	dia.	inches	dia.	inches	dia.
G1	0·08	4·6	57	5·6	70	4·7	59	4·1	51
K	0·2	22	110	27	135	23	113	21	105

From Table 5 it appears that a limiting value for slip of 2·5 per cent of the maximum will bring the equations for the distribution curves into line with the measured values when the initial steel stress is 75 tons per square inch, and the prestress 3,000 lb. per square inch.

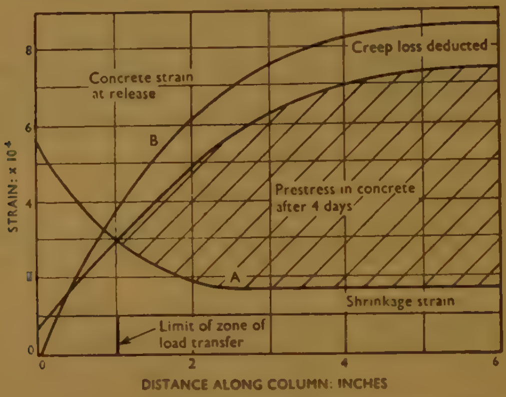
CONDITIONS IN COLUMNS AFTER THE RELEASE OF THE PRESTRESSING FORCE

Slip, Strains, and Bond Stress

Many investigators have noticed that the transmission length does not remain constant for a given member, but increases at a decreasing rate as time goes by. Since the transmission length is a function of the distributions of slip, steel and concrete strains, and bond stresses, it follows that the distribution of all these quantities must change with time. The exponential relation deduced for the distribution of these quantities depends on the existence of a linear relation between slip and steel strain, and *Fig. 4* (*b*) shows that this does not occur when readings are taken after an interval of time.

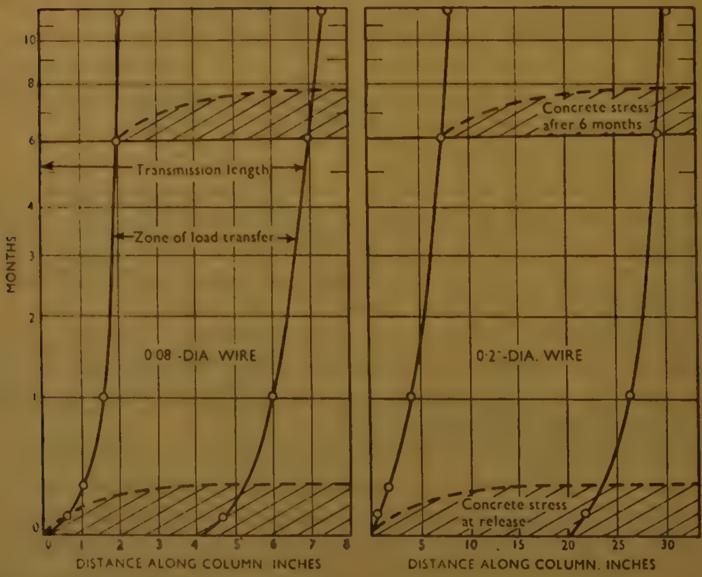
Fig. 3 (*c*) shows that the main action of time effects is to eliminate bond stresses at the end of a column; from this it follows that the steel strain in the end of a column will remain constant at the value associated with zero stress, as seen in *Fig. 3* (*b*). However, the load that was released must still fall upon the concrete, although some of it will be accounted for by creep and shrinkage losses, and the main result of time effects is to move the strain curve bodily towards the centre of the column a distance equal to

Fig. 5



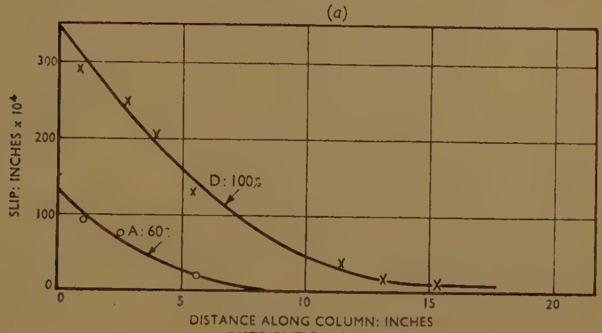
Action of time effects in the transmission length after 4 days (Six 0·08″-dia wires)

Fig. 6

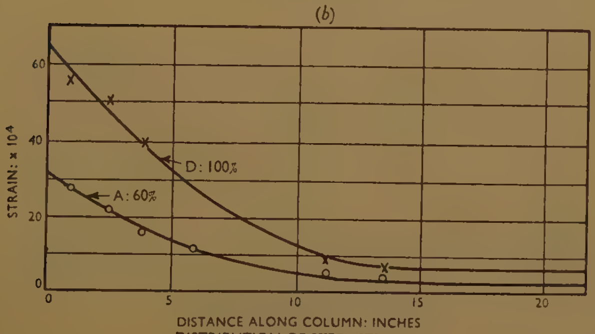


Action of time effects on the transmission length of columns reinforced with 0·08-inch- and 0·2-inch-diameter wires

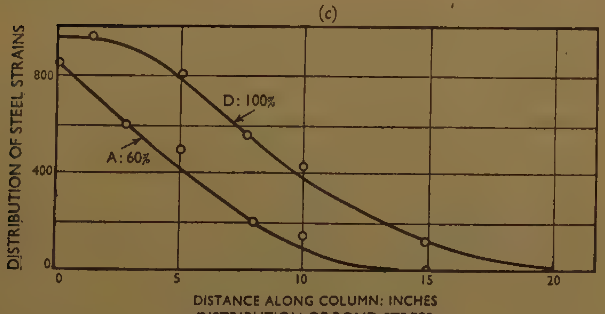
Fig. 7



DISTRIBUTION OF SLIP



DISTRIBUTION OF STEEL STRAINS



DISTRIBUTION OF BOND STRESS

Slip, strains, and stress in column K
(One 0·2″-dia wire)

the change in transmission length. After one month this distance is equal to 1·3 inch, increasing to 1·7 inch after 4 months. Throughout this distance bond stress is zero, but as load starts to come on to the concrete it builds up rapidly to a peak from which it descends gradually. Thus, the end inch of column is completely unstressed after one month, and conditions would not be altered if it were sawn off. In this area of zero stress the slip curve will develop into a straight line of slope equal to the original loss of strain in the steel during transfer.

Fig. 4 (*b*) shows that, even after an interval of time, the relation between slip and steel strains remains linear over the portion of the curve where bond stresses are acting. In this portion, known as the zone of load transfer, the constant C_1 in the relation $\Delta = C_1(S - S_o)$ is increased from 1·20 to 1·68, and from consideration of the foregoing theory it will be seen that this increases the length of the zone of load transfer. Thus, time effects both increase the length of the zone of load transfer, and move it away from the end of the column.

The term "time effects" covers any factor that operates after the release of the prestressing force. Time effects operating in concrete members are creep, shrinkage, and temperature changes.

Creep in steel was to a large extent eliminated by temporarily overstraining the wires before casting. The tension was maintained at a value 7 per cent greater than the working load for 15 minutes, before being relaxed to its normal value.

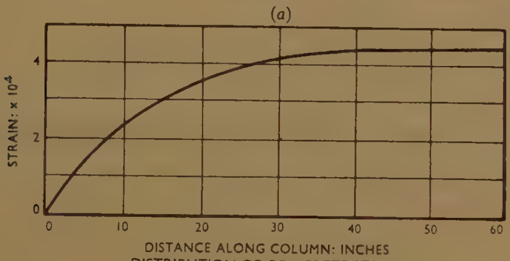
Creep in concrete is proportional to the stress up to about 35 per cent of the crushing stress. Creep in the longitudinal direction is one of the overall losses affecting the final strains in the column, and creep in the radial direction results from radial stresses induced by shrinkage and the wedge effect. Because bond resistance depends on these radial stresses, creep may substantially reduce the bond, especially at the ends of columns where one degree of restraint has been removed.

Shrinkage causes an overall contraction of the concrete, thereby reducing prestress. It is caused by evaporation of water from the concrete and, since this can take place from the ends of the column as well as from the sides, shrinkage strains are temporarily greater in the ends of columns. The determination of these strains is fairly complicated, and proceeds in two stages; an analysis of the distribution of water in the column, and the fitting of a stress pattern to this distribution. Curve A in *Fig. 5* shows the distribution of shrinkage strains in a column after 4 days. Curve B represents the strains resulting from prestress, plotted to the same scale; thus, after deducting the creep loss, the distance between these curves represents the strain associated with a compressive stress in the concrete. From this graph it may be seen how differential shrinkage causes the zone of load transfer to move in from the end of the column. *Fig. 6* summarizes the action of time effects in columns reinforced with 0·08-inch- and 0·2-inch-diameter wires. Differential shrinkage, which causes the "toe" on the load transfer curve, has more significant action in the shorter transmission length. This is because the bulk of the longer transmission lengths are removed from the end of the column where differential shrinkage occurs.

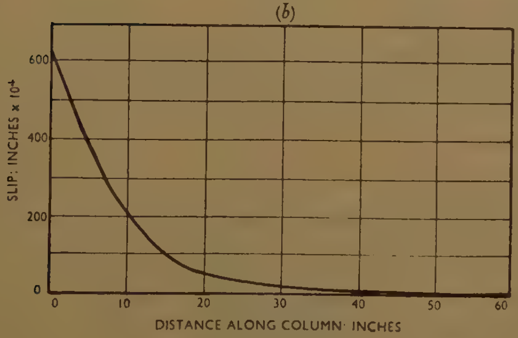
The Significance of End Slip

From *Fig. 3*, it is seen that, whilst time effects cause considerable disturbance to the distribution of strain and bond stress, the distribution of slip remains an ascending curve. The slope of the curve at any point is proportional to the difference between the steel strains at that point and in the centre of the column and, since the steel strains are constant beyond the zone of load transfer, the slip curve becomes a straight line. Now, if the slope of a line is great, a considerable change in this slope will produce little angular change in the direction of the line, as $\underset{m\to\infty}{\mathrm{Lt}}\tan^{-1}m = \frac{\pi}{2}$ where m is the slope of the line. Therefore, a line whose slope is changing

Fig. 8



DISTRIBUTION OF CONCRETE STRAIN



DISTRIBUTION OF SLIP

STRAIN AND SLIP IN COLUMN L
(One 0·275″-dia wire)

approximates to a straight line if its slope is great; and an exponential relation can be applied to the whole of the slip curve, including the straight portion at the end, without introducing serious errors.

Using the notation given under the heading "Distribution of bond effects," the distribution of slip is given by:

$$\Delta = \Delta_1 e^{-C_4 x}$$

differentiating: $$\frac{d\Delta}{dx} = -\Delta_1 C_4 e^{-C_4 x}$$

Now when $x = 0$: $$\frac{d\Delta}{dx} = -(S_1 - S_o)$$

and when $x = l$: $$\frac{d\Delta}{dx} = -(S_1 - S_o)\frac{\Delta_o}{\Delta_1}$$

where the ratio $\frac{\Delta_o}{\Delta_1}$ is used to define the end of the transmission length:

$$(S_1 - S_o) = \Delta_1 C_4 \qquad (x = 0)$$

$$\frac{\Delta_o}{\Delta_1}(S_1 - S_o) = \Delta_1 C_4 e^{-C_4 l} \qquad (x = l)$$

therefore $$\frac{\Delta_o}{\Delta_1} = e^{-C_4 l}$$

now $$C_4 = -\frac{d\Delta/dx}{\Delta} = \frac{(S_1 - S_o)}{\Delta_1}$$

so that $$\frac{\Delta_o}{\Delta_1} = e^{-\frac{(S_1 - S_o)}{\Delta_1}}$$

giving $$\frac{(S_1 - S_o)}{\Delta_1} l = \log_e \frac{\Delta_1}{\Delta_o}$$

therefore $$l = \frac{\Delta_1}{(S_1 - S_o)} \log_e \frac{\Delta_1}{\Delta_o}$$

The ratio $\frac{\Delta_1}{\Delta_o}$ is arbitrary, and has been taken as 40, since Table 5 shows that this gives good agreement with measured transmission lengths. The transmission length is then:

$$l = \frac{\Delta_1}{(S_1 - S_o)} \log_e 40 = \frac{3{\cdot}6\Delta_1}{(S_1 - S_o)}$$

Thus, the transmission length has been expressed in terms of quantities easily measurable. The values for S_1 and S_o may be determined from the extension of the wires and the stress in the concrete respectively. The end slip may be determined by measuring the distance between scratches made on the wire both before and after release by a sharp marker guided by a steel rule held against the concrete. In all the columns tested this formula has given good agreement with the measured results, but it provides only approximate values for twisted wires because these show a departure from the exponential relation.

Bond Effects in Pretensioned Beams

Many of the factors such as slip and bond stress that act in the transmission length of prestressed columns also occur in the vicinity of cracking in bonded beams ; these were investigated by the same technique. Beams were tested beneath the X-ray unit in a loading frame of heavy welded construction, illustrated in *Fig. 9*. The frame was designed to withstand a central load of 5 tons and devices for applying and measuring the load were incorporated. Widths of cracks were measured by a microscope fitted with a graticule reading to 0·0001 inch, and crack patterns obtained for typical beams reinforced with 0·08- and 0·2-inch-diameter wires are given in *Figs 10* (*c*) and *11* (*b*).

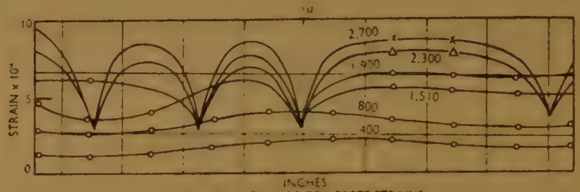
Concrete Strains

The distribution of concrete strains is given in *Fig. 10* (*a*). They were measured in the vicinity of the slot in the lead shielding shown in *Fig. 9* by resistance strain gauges, the gauge factor being determined for each gauge individually by straddling it with an extensometer in the early stages of loading. The zero line for concrete strains is taken as the strain associated with the precompressive stress.

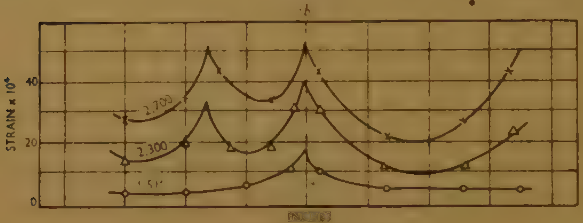
Steel Strains

Steel strains were determined by measuring from the X-ray film, and typical results are shown in *Figs 10* (*b*) and *11* (*a*). In the early stages of loading before cracking occurs, strains in the steel and concrete increase uniformly until a tensile strain corresponding to the modulus of rupture of the concrete is reached. At this point a crack develops in the concrete and concrete strains are redistributed in a manner similar to that occurring in the transmission length of a column. Across the crack steel strain is increased because the tensile reinforcement now carries in addition the load that was previously carried by the concrete. Continued loading increases the concrete strains until the modulus of rupture is attained at another point, and another crack is formed, with consequent release of stress. However, the loops that occur in the steel strain curve indicate that right up to failure the concrete is carrying stresses of a value approaching the modulus of rupture, and this will have an influence on the deflexion curve. Thus, the " cracked section " theory is seen to overestimate the average tensile stresses in the steel, because it ignores stresses in the concrete which may be greater than half the modulus of rupture.

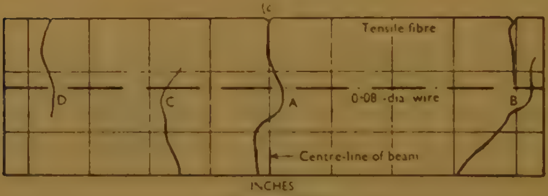
Fig. 10



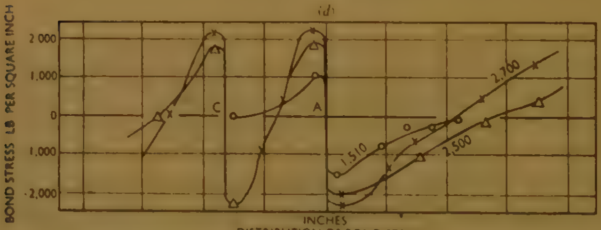
DISTRIBUTION OF CONCRETE STRAINS



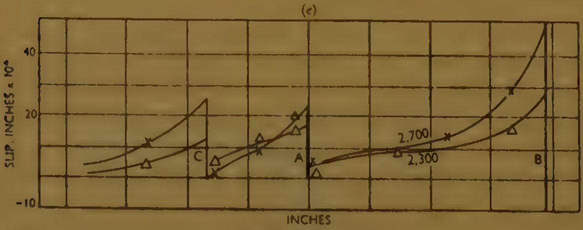
DISTRIBUTION OF STEEL STRAINS



CRACK PATTERN



DISTRIBUTION OF BOND STRESS

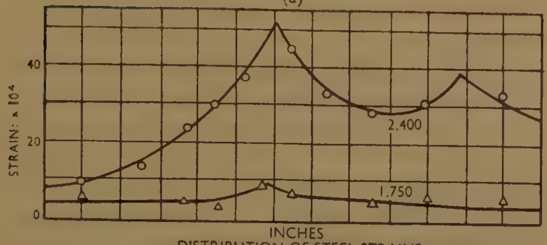


DISTRIBUTION OF SLIP

Beam A4. Slip, strains, and stresses
(Six 0·08″-dia wires)

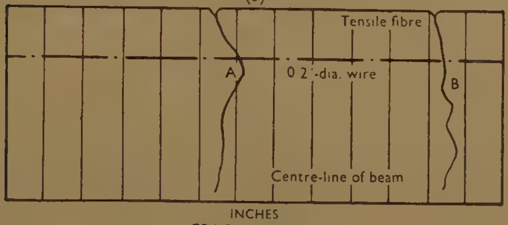
Fig. 11

(*a*)



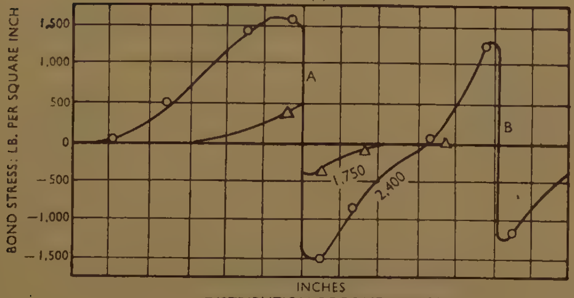
DISTRIBUTION OF STEEL STRAINS

(*b*)



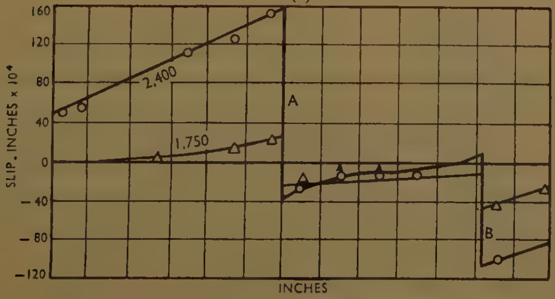
CRACK PATTERN

(*c*)



DISTRIBUTION OF BOND STRESS

(*d*)



DISTRIBUTION OF SLIP

Beam A10
(One 0·2″-dia wire)

Bond Stresses

Distributions of bond stress were prepared from the steel strain curves and *Figs 10* (*b*) and *11* (*c*) give the results for two beams. First, values of zero bond stress were plotted, and values of "strain per inch" were tabulated for a series of points along the beam and, by multiplying these by the secant modulus appropriate to the particular value of steel strain, bond-stress curves were prepared. These form smooth curves and attain maximum and minimum values in the vicinity of cracks. Bond stresses were not detectable until cracking commenced and, as the cracks opened, the values of bond stresses increased. Table 6 shows that bond stresses obtained from beams at working load are in every case greater than those obtained from columns with corresponding reinforcement.

TABLE 6

Reinforcement (in tensile zone for beams)	Bond stresses in beams		Bond stresses in columns	
	Average maximum value : lb. per square inch	Greatest maximum value : lb. per square inch	Average maximum value : lb. per square inch	Greatest maximum value : lb. per square inch
Six 0·08″-dia. plain wires .	2,200	2,500	1,540	1,630
Six 0·08″-dia. wires coated with Araldite . . .	580	600	410	410
One 0·2″-dia. plain wire . .	1,650	1,710	930	950
One 0·2″-dia. indented wire .	2,100	2,100	1,420	1,390
One 0·275″-dia. wire . .	—	—	920	920

These high bond stresses suggest the action of tangential friction, since this quantity depends only on the crack width and is unaffected by disturbed conditions in the concrete. Because tangential friction increases with increased stress in the steel, its contribution in prestressed concrete is much greater than in ordinary reinforced concrete. Its complete absence in the pull-out column reduces the validity of this analogy for conditions in the section of a prestressed beam between the end crack and the support. Abrams and Mylrea deduced that very high bond stresses would occur in the vicinity of cracks, but these stresses are very localized and it is thought that they may have been made apparent only because steel strains were measured for gauge lengths as small as 0·3 inch. Mylrea deduced from theory that the maximum value of bond stress would occur at a short distance from the crack, but he did not take into account the action of tangential friction, and maximum values were recorded nearer the crack surface than he had anticipated.

Slip

Distributions of slip are given in *Figs 10* (*e*) and *11* (*d*). Movements of steel to the right, relative to the concrete, are plotted as positive, and movements to the left are negative. At the edges of a crack the concrete slides back to expose the steel, thereby causing a positive slip at the left of the crack and a negative slip at the right, and the total ordinate between these two curves is the width of the crack. Thus, the slip curves, which were readily obtainable, provide direct information as to the distribution and size of cracks. In the early stages of loading slip occurs only in the vicinity of cracks, and forms a curve of exponential shape. Now it has already been stated that the localized height of peaks on the steel strain curve is limited by the modulus of rupture of the concrete ; therefore, for a given crack pattern the whole curve will tend to be displaced by increased load with little alteration in its shape. Thus, the percentage variation of strain is reduced as the strain increases, and the strain becomes more uniform. Now the slope of the slip curve is proportional to the steel strain, so it follows that with increasing load the slip tends to develop into a series of broken straight lines.

From consideration of the geometry of the slip curve when it has reached this condition, crack width is seen to be proportional to the slope of the slip curve and therefore to the average steel strain, and this will cause the deflexion curve to be a straight line for a given well-developed crack pattern. It is difficult to obtain results of value with regard to crack spacing because the zone of cracking was limited. Thus, Table 7 gives an infinite spacing of cracks for beam A11 since this failed with one central crack.

TABLE 7

Beam	Load : lb.	Conditions at a total crack width of 0·025 inch			
		Width of central crack : inch	Number of cracks in beam :	Average spacing : inches	Bond stress at central crack : lb. per square inch
A 4	2,230	0·007	5	1·8	2,500
A 7	2,110	0·005	6	2·1	2,400
A 8	2,080	0·013	4	6·2	590
A10	2,320	0·016	2	5·5	1,690
A11	2,200	0·025	1		1,520
A12	2,280	0·010	5	2·5	2,200

It does, however, illustrate a relation between bond stress and crack spacing. Bjuggren[9] found a linear relation between the spacing of cracks and their width for a given bond stress, and the tabulated values for bond stresses of the order of 1,600 and 2,200 lb. per square inch give some measure of support to this. It is anticipated that a statistical analysis will be necessary in order to discover some basic order behind the seemingly erratic cracking of a concrete beam, and this will require the testing of many more specimens, in a research programme for which this Paper is merely the preliminary investigation.

SUMMARY

(1) Bond resistance results from friction. The quality commonly referred to as adhesion arises from frictional effects recurring on the microscale and is, therefore, destroyed by slips occurring on the macroscale. Slip and bond stress always appear together.

(2) The " adhesive " factors of bond resistance are micromechanical locking and molecular attraction. The " frictional " qualities are frictional resistances from shrinkage pressures, mechanical locking, dilatency, and wedge action.

(3) Tangential friction is an important source of bond resistance in the cracked beam. This is proportional to $\frac{Mw}{2a^2}$ where M is the external bending moment, w the width of the crack, and a the lever arm.

(4) The maximum bond stress recorded in beams was about 2,500 lb. per square inch and in columns about 1,700 lb. per square inch; the bond stresses observed in beams being always greater than those found in the transmission length of columns with the same reinforcement.

(5) On release of the prestressing force, the slip, steel strains, concrete strains, and bond stresses for a pretensioned member are distributed in the transmission length according to an exponential relation. Time effects (creep and shrinkage) invalidate this relation, causing the transmission length to increase both by increasing the length of the zone of load transfer and by moving in from the end of the column.

(6) Except for twisted wires the formula $l = \frac{3{\cdot}6\Delta_1}{(S_1 - S_o)}$ gives the transmission length with good accuracy, both for instantaneous release and at times after this. Δ_1, is the end-slip or ' pull-in ' of the wires, S_1 is the initial strain in the wires, S_o is the final strain in the concrete. For twisted wires the accuracy is fair.

(7) Grease, rusting, notching, and indentations have a great effect on bonding efficiency. The thickness of a grease film provides a convenient standard for degree of greasiness.

(8) Large local tensile and bond stresses occur in the tensile reinforcement of pretensioned beams in the vicinity of cracks. These stresses are many times greater than those calculated by the traditional or conventional shear formula and they may occur in regions of pure bending.

(9) It is recommended that about one-half of the transmission length should overhang the support in a simply supported beam. Where there is end fixing the whole of the transmission length should overhang. This limits the diameter of wire that may be used for pretensioned work.

(10) High ultimate bond strengths reduce both the width of cracks and their spacing.

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The Paper, which was received on the 11th May, 1954, is accompanied by three photographs and eight sheets of drawings, from which the half-tone page plates and the Figures in the text have been prepared.

CORRESPONDENCE on this Paper should be forwarded to reach the Institution by the 15th July, 1955. Contributions should not exceed about 1,200 words.—SEC.

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Wheildon Brown, Timothy, B.A. (*Cantab.*), Grad.I.C.E.
Williams, Antony Clive, B.Sc. (*Birmingham*).
Wood, John Frederick Dilworth, Grad.I.C.E.
Woodburn, George, B.Sc. (*Edinburgh*), Stud.I.C.E.
Woodward, Peter, Grad.I.C.E.
Wroe, Donald Arthur, B.Sc.Tech. (*Manchester*), Grad.I.C.E.
Wylie, David, Stud.I.C.E.
Yeadon, Harry Leslie, B.Sc.Tech. (*Manchester*).
Young, Clifford, B.Sc. (*Manchester*).

DEATHS

It is with deep regret that intimation of the following deaths has been received.

Members

Alan Boothman (E. 1929, T. 1943).
Robert James Boyd, M.E. (E. 1907, T. 1922).
George Bransby-Williams (E. 1897, T. 1909).
George Edward Bohun Coulcher (E. 1910, T. 1926).
Frank Harding Jones (E. 1909).
Henry Abraham Smith, B.Sc. (E. 1920, T. 1938).
Kenneth Maurice Steven, B.Sc. (E. 1931, T. 1948).

Associate Members

Hugh Ritchie Barr (E. 1905).
Frederick Francis Percival Bisacre, O.B.E., M.A., B.Sc. (E. 1914).
Patrick Cullinane, B.E. (E. 1949).
Mathrage Romiel Dharmasiriwardhane (E. 1932).
Charles Garrod, B.Sc. (E. 1904).
Thomas Daniel Hamilton, M.C. (E. 1914).
Edward Townsend Hippisley, M.A. (E. 1941).
Arthur Daniel Jaffe, O.B.E., M.A. (E. 1905).
William Stevenson MacCulloch (E. 1908).
Arthur Neville Marchant (E. 1934).
Maxwell Robert Puddy, B.Sc. (E. 1950).
Percival Wilberforce Turner (E. 1906).
Fred Curry Walls (E. 1929).
Arthur Trevor Yeo West (E. 1946).

OBITUARY

ROBERT BAKER, O.B.E., who died at his home in Burley, Hampshire, on the 29th July, 1954, was born at Barkston Ash, Yorkshire, on the 11th August, 1874. After three years' training in the manufacture of instruments he was articled for three years to Sir Robert Elliott-Cooper, and then embarked on a varied career of railway engineering, serving in England, Greece, Malaya, and China from 1911 until his retirement in 1934. He was concerned mainly with the administration and maintenance of the Kowloon–Canton Railway in Hong Kong, filling the post of Chief Engineer from 1927 onwards, combining that post with Manager. During that time he did some pioneer work in the development of reinforced concrete sleepers.

Mr Baker was elected a Member of the Institution in February 1915; he was also a Fellow of the Permanent Way Institution. The Order of the British Empire was bestowed upon him in 1933.

He is survived by his only daughter who is married to The Hon. Mr Justice Anthony Somerhough, O.B.E., Q.C., of the High Court of Northern Rhodesia.

PERCIVAL ARTHUR RUPERT LEITH, who died on the 8th September, 1954, was born on the 10th December, 1888.

He received his early education at Robert Gordon's College, Aberdeen, and his engineering training under R. Gordon Nicol, Engineer to the Aberdeen Harbour Commission.

From 1906 to 1912 he was an engineer on public works contracts. In 1913 he served as Civil Engineering Assistant in Rosyth Dockyard. In 1914 he was appointed Chief Civil and Mechanical Engineer to Smith's Dock Co. Ltd, North Shields.

From 1926 until his retirement in 1953 he was Chief Engineer to the Tees Conservancy Commissioners.

Mr Leith was elected a Member in 1932.

He is survived by his wife, Mrs E. M. Leith.

LESLIE PRESTON PARKER, O.B.E., B.Sc.(Eng.), who died at Chigwell Row on the 23rd September, 1954, was born on the 22nd October, 1888.

He was educated at Bancroft's and served his apprenticeship with the former Great Eastern Railway Co., at Stratford Locomotive Works.

Subsequently he held various posts with the London and North Eastern Railway Co., becoming District Locomotive Superintendent at Stratford, and later Locomotive Running Superintendent (Eastern) at Liverpool Street Station, London.

At the time of his death he was Motive Power Superintendent (Eastern Region) British Railways.

Mr Parker was elected an Associate Member in 1918, and was transferred to the class of Members in 1946. He was also a Member of the Institution of Mechanical Engineers, and an Associate Member of the Institution of Locomotive Engineers.

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